STRUCTURE SELECTION REPORT

EAST PAPAGO FREEWAY - SEGMENT 6

SALT RIVER BRIDGE

202L MA 151 H 2151 01C

EAST PAPAGO
(INDIAN BEND WASH-LOOP 101)

Prepared by :

T.Y. LIN INTERNATIONAL
1817 North Seventh Street, Suite 175
Phoenix, Arizona 85006

Submitted By :

GANNETT-FLEMING of ARIZONA, INC.
7975 North Hayden Road, Suite A-105
Scottsdale, Arizona 85258

DRAFT Aug. 1989

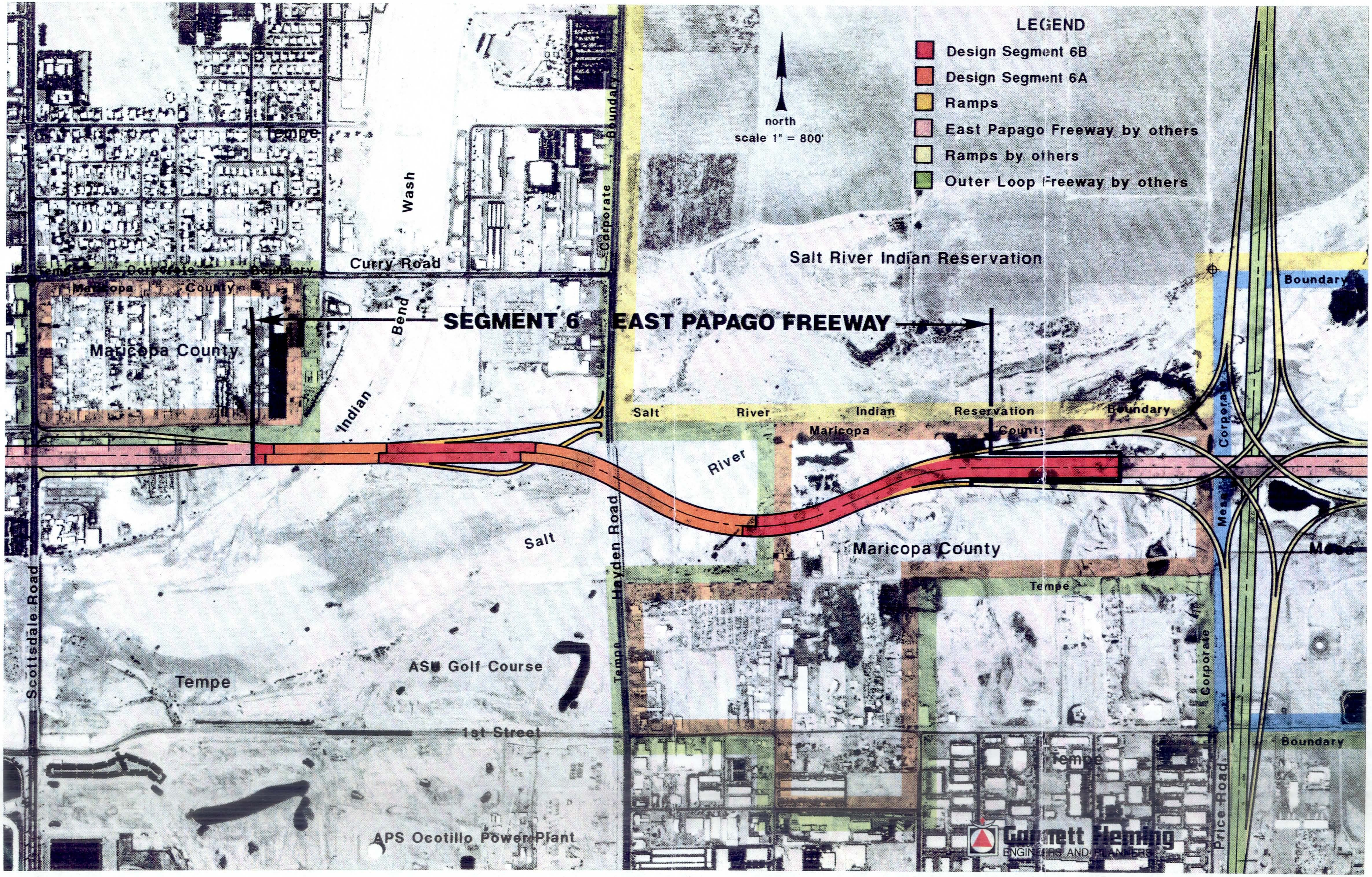
TABLE OF CONTENTS

FIGURES

Figures

TABLES

Tables



LEGEND
Design Segment 6B
Design Segment 6A
Ramps
East Papago Freeway by others
Ramps by others
Outer Loop Freeway by others
north
scale 1" = 800'
SEGMENT 6 EAST PAPAGO FREEWAY
Salt River Indian Reservation
Salt River Indian Reservation Boundary
Maricopa County
Curry Road
Indian Bend Wash
Salt River
Scottsdale Road
Hayden Road
Price Road
Tempe Corporate Boundary
Mesa Corporate Boundary
Maricopa County
Tempe
ASU Golf Course
1st Street
APS Ocotillo Power Plant
ENGINEERS AND PLANNERS

Figure 1

EXECUTIVE SUMMARY

This report presents the results of comparative cost studies for various alternative bridge types and span lengths for the East Papago Freeway Bridge over the Salt River at Hayden Road.

This segment of the East Papago Freeway consists of four lanes plus an HOV lane in each direction separated by a concrete safety barrier in the median. The normal 22-foot wide median is transitioned to 27-feet prior to the Salt River Bridge to provide for separate eastbound and westbound structures. The bridge begins east of Indian Bend Wash crossing over the off-ramp, Hayden Road and the Salt River on a 3^{0}-30' reverse curve alignment. The eastbound bridge is 2621 feet long and the westbound bridge is 2426 feet long. The total deck area is 419,742 square feet.

The constraints to the bridge layout are the off-ramp (Ramp B), Hayden Road and the underground utilities located in a corridor crossing the river east of Hayden Road. Two basic span length layouts were developed - a long span layout based on an economic span length study and an AASHTO girder layout using maximum length Type VI precast girders.

The potential effect of scour on the underground utilities and the Hayden Road Bridge foundation was studied and the risk evaluated. Various numbers and sizes of columns and drilled shaft foundations were analyzed to determine the optimum pier configuration.

Foundation designs were based upon preliminary drilled shaft capacities developed by Sergent, Hauskins and Beckwith from three borings and confirmed by three other borings 150 feet deep. All borings were located in the river east of Hayden Road. There is no geotechnical information available for the 40% of the bridge located west of Hayden Road because of the lack of access to that property. Foundations for the piers will consist of single drilled shafts from 8' to 10' in diameter supporting each column of a two-column pier. These shafts will vary between 110 and 175 feet deep.

Preliminary designs were performed for six types of bridges as follows:

Two Cast-in-Place Box Girder Schemes

Two Precast Drop-in Type Schemes

Curved Steel Plate Girders

AASHTO Type VI Girders

In addition, comparative estimates were prepared for different size columns and drilled shafts and for Type VI Modified girder spans.

The cost-versus-span length studies indicated that span lengths from 175 feet to 200 feet were the economic range for both concrete box girders and steel plate girders. The total bridge estimates were developed from preliminary design quantities for typical 3 and 4 span units and expanded according to the span layouts for each bridge type. Unit prices were developed from discussions with contractors, suppliers, fabricators and analysis of recent bid prices in the Phoenix area.

The two precast drop-in schemes (Alternates C and D) were deleted by consensus due to the number and frequency of expansion joints and potential rideability problems. The twin post-tensioned box girder alternate (B) proved to be superior to the single box girder alternate (A) in both economy and constructability.

Our alternative analysis of the final three bridge types revealed that total costs were approximately equal with the cast-in-place box girder layout constructed on erection trusses. The estimated cost of the box girders (Alternate B) constructed on falsework is 10% less than the steel girders (Alternate E) and 11% less than precast AASHTO girders (Alternate F).

The twin box girder alternate best satisfies the objective of providing an aesthetically pleasing bridge with fewer columns and a proven low maintenance cost record in the Phoenix area. This type of bridge is also estimated to be more economical than either steel or precast AASHTO girders if constructed on conventional falsework. Construction over Hayden Road on falsework will cause some disruption to existing traffic which would be minimized with either steel or precast girders.

After consideration of these different bridge types, span layouts and constructability we recommend that the twin post-tensioned concrete box girders supported by twin column piers founded on single drilled shafts be selected for this East Papago Salt River Bridge. It is also our recommendation that Special Provisions be developed to control the amount of falsework permitted in the river at any time and to prescribe protective measures to be taken by the contractor. Such specifications would be based upon historical records of river flow with the amount of vulnerable falsework varied according to seasonal risk.

1. INTRODUCTION

1.1 Description of Project

The East Papago Freeway is the East-West link between the Papago Freeway from the Squaw Peak Parkway to the Outer Loop Interchange connecting with the Pima, Price and Red Mountain Freeways. It also provides access to Sky Harbor Airport via the Hohokum Expressway and Sky Harbor Boulevard.

Segment 6 of the East Papago Freeway begins just west of Indian Bend Wash and terminates west of the Outer Loop Interchange for a total length approximately 6500 feet. The proposed alignment continues easterly from across Indian Bend Wash and the eastbound off-ramp to Hayden Road, over Hayden Road, turning southeasterly to cross the Salt River. The alignment then turns easterly and continues along the south bank of the river to connect with the Outer Loop Interchange. The Freeway is a 10-lane limited access highway in this segment with eastbound-off and westbound-on connecting ramps at Hayden Road.

For purposes of design and construction, this Segment is divided into two contracts. Contract 6A consists of bridges over Indian Bend Wash and the Salt River, retaining walls, and some river bank protection. Contract 6B includes the embankment, paving, connecting ramps to Hayden Road, storm drains, utilities, traffic control and minor drainage.

The Salt River Bridge will be the largest, most prominent and most expensive structure on the East Papago Freeway. The purpose of this report is to summarize the progress to date in developing possible bridge types and span configurations which result in an aesthetically pleasing, constructable and economical structure.

1.2 Preliminary Meetings and Investigations

During the development of the study alternatives, we participated in several meetings in an information-gathering process. The objectives of the meetings were to:

* Further refine the scope of the study by reducing the number of alternatives to those which are most economically practical and constructable.
* Determine any restrictions that will be on construction methods.
* Establish realistic unit prices to use for the estimates.
* Determine the practical depth and size limitations on drilled shaft construction.
* Define the depth and extent of scour at the piers.
* Determine foundation capacities.

These meetings included representatives from:

* ADOT, Structures Section
* Management Consultant - DMJM
* Three drilled shaft contractors
* Two general contractors
* Simons, Li & Associates, Inc.
* Sergent, Hauskins & Beckwith
* American Institute of Steel Construction
* Prestressed concrete suppliers
* Structural steel fabricators
* Expansion joint suppliers
* Utility owners

1.3 Site Investigations

We also conducted several site investigations to identify potential problem areas and take photographs and notes for future reference. We included an evaluation of the existing Hayden Road Bridge and as-built plans. Pertinent record drawings for this Maricopa County owned bridge are included in the Appendix - Section 12.

Due to the proximity of the new piers to the existing 6' diameter drilled shafts, the effect of scour on the existing Hayden Road Bridge has been considered in these preliminary studies. It will be necessary, however, to investigate this further upon final selection of bridge type and layout.

1.4 Scope of the Report

The scope of this major structural report includes the preliminary layouts of both the eastbound and westbound bridges, including line and grade, number and length of spans, type of structure and critical dimensions and clearances. Potential competitive bridge types of concrete and structural steel were studied, configuration and governing design parameters established, construction methods investigated and approximate costs estimated.

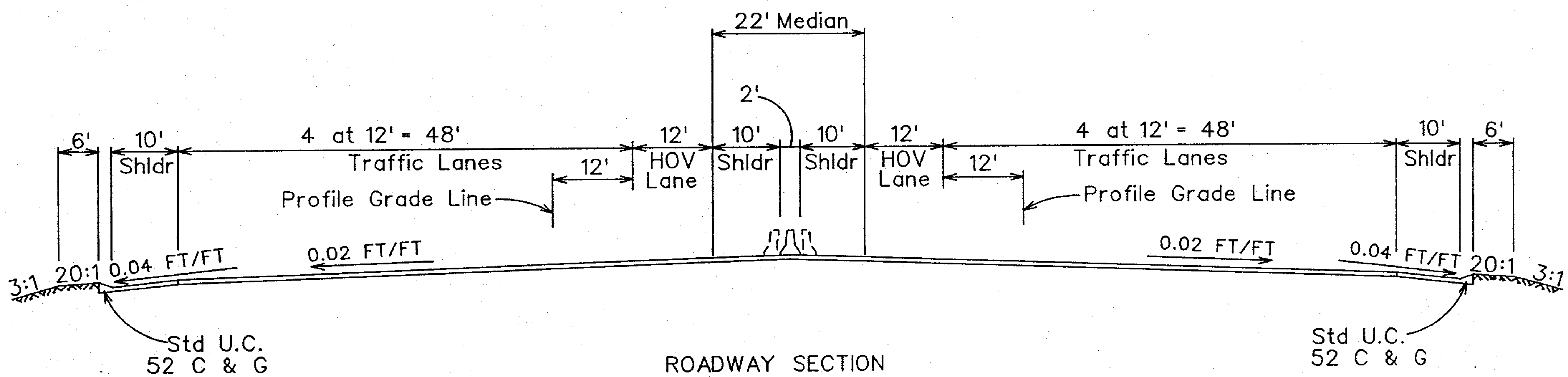
2. BRIDGE GEOMETRICS

The East Papago Freeway through Segment 6 consists of four 12-foot traffic lanes and a 12-foot High Occupancy Vehicle (HOV) lane with 10-foot shoulders, in each direction, separated by a concrete traffic barrier in the median. This is the ultimate width of roadway with no provisions for future widening. The normal 22-foot median width will be transitioned to 27-feet prior to the Salt River Bridge to accomodate similar profile grades on the Eastbound and Westbound structures, with two independent traffic barriers and construction clearance throughout the two superelevated curves. Typical roadway and bridge cross-sections are shown in Figure 2.

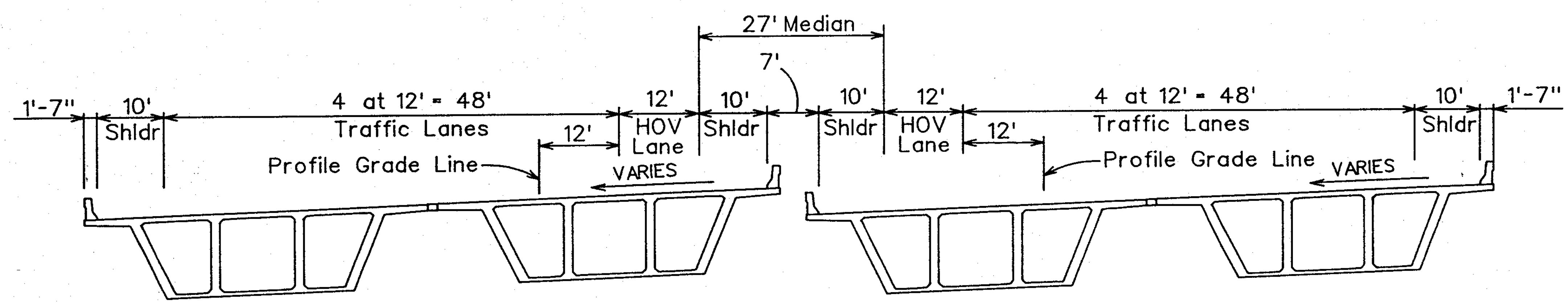
The horizontal alignment of the Salt River Bridge consists of reverse 3^{o}-30' curves separated by an 800-foot long tangent. Minimum vertical clearances of 16'-6" will be provided over the Ramp B off-ramp and the Hayden Road Bridge. The curvilinear alignment requires a maximum superelevation of 0.06 ft/ft. which results in a differential elevation of

4.8' across each of the two 80-foot wide bridges. The skew is approximately 60^{o} right and 40^{o} left with Hayden Road. The length of bridge is dictated by Ramp B on the west end and the river bank on the east end. The west abutments for the two roadways are staggered to reduce the length of the westbound roadway bridge. A short retaining wall is provided between the staggered abutments along the median. A retaining wall will also be required between the eastbound roadway and Ramp B.

TYPICAL SECTIONS



ROADWAY SECTION



BRIDGE SECTION

FIGURE 2

3. UTILITIES

There are major utilities crossing the bridge site both above and below ground.

3.1 <u>Above Ground</u>

Arizona Public Service (APS) has four (4) 230KV transmission lines located approximately 140' west of Hayden Road that cross the Salt River on the alignment at about Sta. 295. These lines are supported by steel towers located on each bank. It will be necessary to raise or relocate these lines prior to construction. APS currently plans to install permanent steel poles at the south right of way line which will raise the lines high enough to provide proper construction clearances. These lines could be lowered to the minimum required over traffic following construction.

APS also maintains a 69KV and a 12KV line supported by wooden poles located just west of and parallel to the Hayden Road Bridge. Underslung from this line are Salt River Project (SRP) communication lines. These lines will have to be deactivated at certain times during construction. The owners are considering permanent disposition alternatives for these lines. APS intends to shoofly the 69KV line west around the bridge construction and then to return it to essentially its present location but at a slightly lower elevation under the new bridge. The SRP lines will be relocated onto the existing Hayden Road Bridge. Other SRP 69KV lines presently located near the Indian Bend Wash may be relocated parallel to the other wire utilities at Hayden Road, but they may be on their own poles. See Figure 4 for locations of all utilities.

3.2 Below Ground

Underground utilities also cross the Salt River on both sides of the Hayden Road Bridge. The City of Tempe has a 3'6" water main that crosses the alignment at about Sta. 296 some 30' west of Hayden Road. The top of this line has an elevation of about 1140 across the proposed alignment.

Five other underground utility lines lie in a corridor located from 86' to 126' east of the centerline of Hayden Road as follows:

Utility	Owner	Offset from Hayden Road	Elevation
36" Sewer	Mesa	88'	1146
18" & 21" Sewer Siphon	Mesa	103'	1127
48" Water	Phoenix	115'	1129
12" 400 psi Ga	SWG	125'	1125

Due to the high cost of relocating these underground utilities, piers on all of the alternate layouts were located to avoid these lines. The Tempe water main will have to be relocated as noted under Bridge Substucture. Consideration has been given to the effect of potential scour on the utilities caused by the piers. This is discussed in further detail in the Bridge Substructure section.

UTILITY LOCATIONS

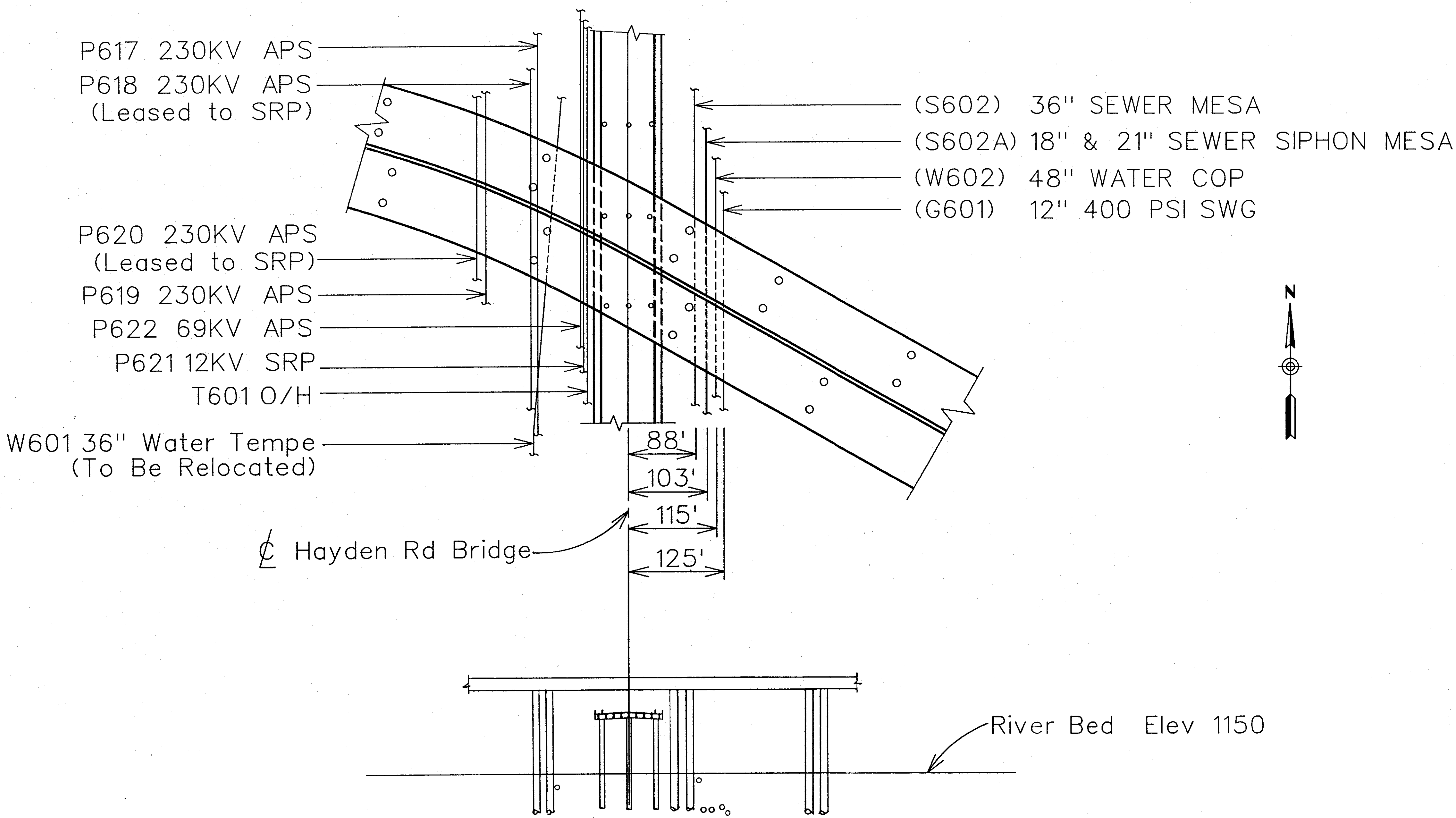


FIGURE 4

4. DRAINAGE CONSIDERATIONS

4.1 Drainage

Flow in this reach of the Salt River is controlled by a series of dams upstream operated by the Salt River Project. A list by dates of average daily spills from the Granite Reef Dam and major drains above Hayden Road are shown in Table 5 in the Appendix - Section 12.3.. This list, from records of the Salt River Project, covers the period from August 1, 1964 thru June 24, 1989.

Hydraulic parameters for this crossing are given in Table 1. The discharges used in the hydraulic analysis are based on values presented in the May 1982 Central Arizona Control Study (U.S. Corps of Engineers.) The 100-year frequency analysis used a design discharge of 215,000 cfs. The standard project flood analysis used a design discharge of 289,000 cfs. The water surface elevation at the upstream face of the Hayden Road Bridge is 1173.0 for the 100-year event and 1176.3 for the standard project flood. Corresponding velocities are 8.8 and 9.9 fps, respectively. The Hayden Road Bridge is 1190' long. The channel at this location is approximately 1000' wide.

Table 1
Hydraulic Parameters
East Papago Freeway Crossing

Project[1] Station (ft)	100-Year Event WSEL (ft)	100-Year Event Velocity (fps)	Standard Project Flood WSEL (ft)	Standard Project Flood Velocity (ft)
36263	1170.7	11.2	1173.4	12.9
36660	1171.5	11.3	1174.4	12.9
36821	1172.4	8.9	1175.4	10.5
36831	1172.6	8.9	1175.6	10.4
36982	1172.8	8.6	1175.9	10.1
36992	1173.0	8.5	1176.1	10.0
37027	1173.0	8.8	1176.3	9.9
37116	1173.7	8.5	1177.0	9.6
37265	1174.2	7.3	1177.4	8.6
37275	1174.3	7.2	1177.5	8.5
37402	1174.3	7.8	1177.5	9.3
37412	1174.4	7.8	1177.7	9.2
37535	1174.5	7.8	1177.8	9.2
37545	1174.5	7.8	1177.9	9.1
37672	1174.6	7.7	1178.0	9.0
37682	1174.7	7.7	1178.1	9.0
37813	1174.8	7.6	1178.3	8.9
37823	1174.9	7.6	1178.4	8.9
37980	1175.0	7.6	1178.5	8.9
37990	1175.1	7.5	1178.6	8.8
38147	1175.1	8.0	1178.6	9.3
38157	1175.2	8.0	1178.7	9.3
38236	1175.4	7.5	1179.3	8.0

[1] Project Station 37027 is the upstream face of the Hayden Road Bridge

4.2 Scour Estimates

4.2.1 Piers

Scour estimates for general and local scour for various pier configurations and foundation types were computed by Simons, Li and Associates, Inc.

Three different pier configurations were examined:

1) piers with three 6-foot diameter columns per structure
2) piers with two 8-foot diameter columns per structure
3) piers with one 15-foot diameter column per structure

The effects of piers with two 10-foot diameter columns were estimated from the results on the 8' and 15' diameter columns. The piers in each case were assumed to be on 200-foot centers and skewed sufficiently to expose each column to the flow.

Computed 100-year water surface elevations for each case are given in Table 2. The results show that the maximum water surface profiles occurs with the use of the 6-foot diameter piers, but the variation among all configurations is only 0.1 feet. This small difference is due to the similar projected width of the different pier configurations when skew is considered; 18 feet for the three 6-foot diameter piers, 16 feet for the two 8-foot diameter piers, and 15 feet for the single 15-foot pier.

The results of the local scour estimates for each pier configuration are given in Table 3. Included in the table are estimates of scour with a 6-foot debris blockage considered at each pier. The estimates given are the most conservative of several pier scour equations reported in the literature and used for this analysis. None of the equations explicitly account for armoring during the scour process, which could limit the depth of scour. Although we routinely consider the armoring process in our sediment routing studies, we are unaware of an adequate means of considering the armor potential for local scour around bridge piers.

The above scour estimates assume drilled cylindrical piers to a depth below the scour hole. Drilled cylindrical piers have been used with success on alluvial streams in Arizona where local scour potential is very high. In order to obtain the total scour depth for each pier configuration, 6 feet should be added to the local scour depth. This additional depth is to account for general scour and bed forms.

The potential scour for piers placed on piles with a 34-foot diameter pile cap was also investigated. Due to the large local scour depths computed, it would appear impractical to place the pile caps below the potential scour depth. There is no method presently available to estimate scour around a pile cap located within the scour zone. It is possible that the pile cap may act as a scour arrestor, blocking the horseshoe vortex and reducing the depth of scour. However, if the pile cap were exposed to the flow sufficiently, it is possible that local scour would be increased due to the additional flow restriction. Using a width of

34 feet in the pier scour equations results in a scour depth of 50 feet. While it is unlikely this depth would be obtained, the potential for local scour is significant. A good estimate of the depth of scour for this situation could only be determined by a physical model study. We believe scour of this potential depth is unacceptable and therefore single drilled shafts directly under the columns are recommended for all alternates.

4.2.2 Underground Utilities

The effect of local scour around the piers on underground utilities and the existing Hayden Road Bridge piers has been evaluated. Piers for the new bridge have been located to minimize disruption of the utilities.

Based upon approximate scour parameters provided by Simons & Li, sections were plotted for the 145-foot spans and 200-foot spans. These sections indicate that the 18" & 21" Sewer Siphons (Mesa), the 48" Water (Phoenix), and the 12" 400 PSI gas (SWG) are below the anticipated scour envelope. The 36"

Sewer (Mesa), a gravity line that presently carries an estimated 8 million gallons per day, is within or close to the anticipated scour envelope. This shallow line, even without the proposed bridge, is susceptible to scour impact. According to information from the City of Mesa the 18" & 21" Sewer Siphons are the back-up system in the event scour damages the 36" gravity sewer. On this basis consideration should be given to leaving this line in place and making provisions for replacement in the area of the proposed bridge should scour damage the 36" gravity sewer.

The 36" water line on the west side of the Hayden Road Bridge is also within the anticipated scour envelope and relocation for approximately 400 feet will be required. The estimated cost to relocate this line is about $90,000 (400 LF @ $225 per LF).

4.3 Proposed Improvements

The City of Tempe is considering channel improvements in their current Rio Salado planning. Consideration is being given to installing grade control structures, bank protection and changes in river bottom slope. These improvements would all be downstream of the Hayden Road Bridge and are expected to have minimal impact on this project. We will, however, continue to monitor their planning in this area.

4.4 Bank Protection

Bank protection will be necessary to protect the embankment for Ramp B on the north bank and from the Hayden Road Bridge through the east abutment location on the south bank. The exact type of bank protection has not been determined at this time.

4.5 Bridge Deck Drainage

The bridge deck will be drained through scuppers in the deck. These scuppers will be located along the gutter line of the low barrier except over Ramp B and Hayden Road.

Table 2. Water-Surface Elevations for 100-Year Peak Discharge, East Papago Freeway Crossing.

River * Distance (ft)	Water-Surface Elevation 6-ft Piers (ft)	8-ft Piers (ft)	15-ft Piers (ft)
0	1,172.44	1,172.44	1,172.44
206	1,172.94	1,172.91	1,172.90
295*	1,173.70	1,173.67	1,173.66
615	1,174.33	1,174.31	1,174.29
1,015	1,174.84	1,174.78	1.174.75
1,415	1,175.35	1,175.28	1,175.25
1,815	1,175.50	1,175.43	1,175.40
2,235	1,175.77	1,175.70	1,175.67
2,635	1,175.78	1,175.71	1,175.68
3,035	1,175.82	1,175.76	1,175.73
3,445	1,176.04	1,175.99	1,175.96
3,855	1,176.97	1,176.93	1,176.90
4,255	1,178.51	1,178.48	1,178.46
4,755	1,180.90	1,180.88	1,180.87

* River Distance 295 is the upstream face of the Hayden Road Bridge.

Table 3. Summary of Pier Scour Estimates East Papago Freeway Crossing

Pier Diameter (ft)	Local Scour (ft)	Local Scour With 6-ft Debris Blockage (ft)
6	16	25
8	19	27
15	29	36

5. SPAN LAYOUT STUDIES

5.1 Unit Cost Curves

The economic span length curve (Figure 9), was developed for the multiple web post-tensioned box girder type bridge. Preliminary analysis and design was performed for a typical continuous unit. The superstructure loads and material quantities from this modular design were projected for the total bridge length to evaluate the various substructure alternatives considered. These included both two column and single column piers. Foundations consisted of single drilled shafts under each column and multiple drilled shafts supporting a pile cap. The various structure alternates are discussed in more detail in Sections 6 and 7.

Figure 9 shows the relative cost per square foot of the superstructure and two column bents versus span length. (Common items as abutments, barriers, expansion joints and bearings were not included.) The sum of these curves indicates the variation in unit cost of the bridge per span length.

The information indicates that using the available geotechnical data, preliminary design, and unit costs assumed for the study. there is very little variation in cost per square foot (less than 1%) in the range of span lengths from around 175' to 200' The offsetting effects of reduction in substructure quantities and costs and increase in superstructure costs yields a minimum of $54.44 per square foot at 200' spans. The slight difference in unit cost for shorter spans, which will be required where other geometric limitations dictate, will have very little effect upon total structure cost.

A similar study was performed for the steel plate girder alternate by the American Institute of Steel Construction (AISC). Span lengths from 140 feet to 260 feet were considered in 5-span units. This information was adjusted for substructure costs to determine the economic span length. The results were similar to that shown for the concrete box girders.

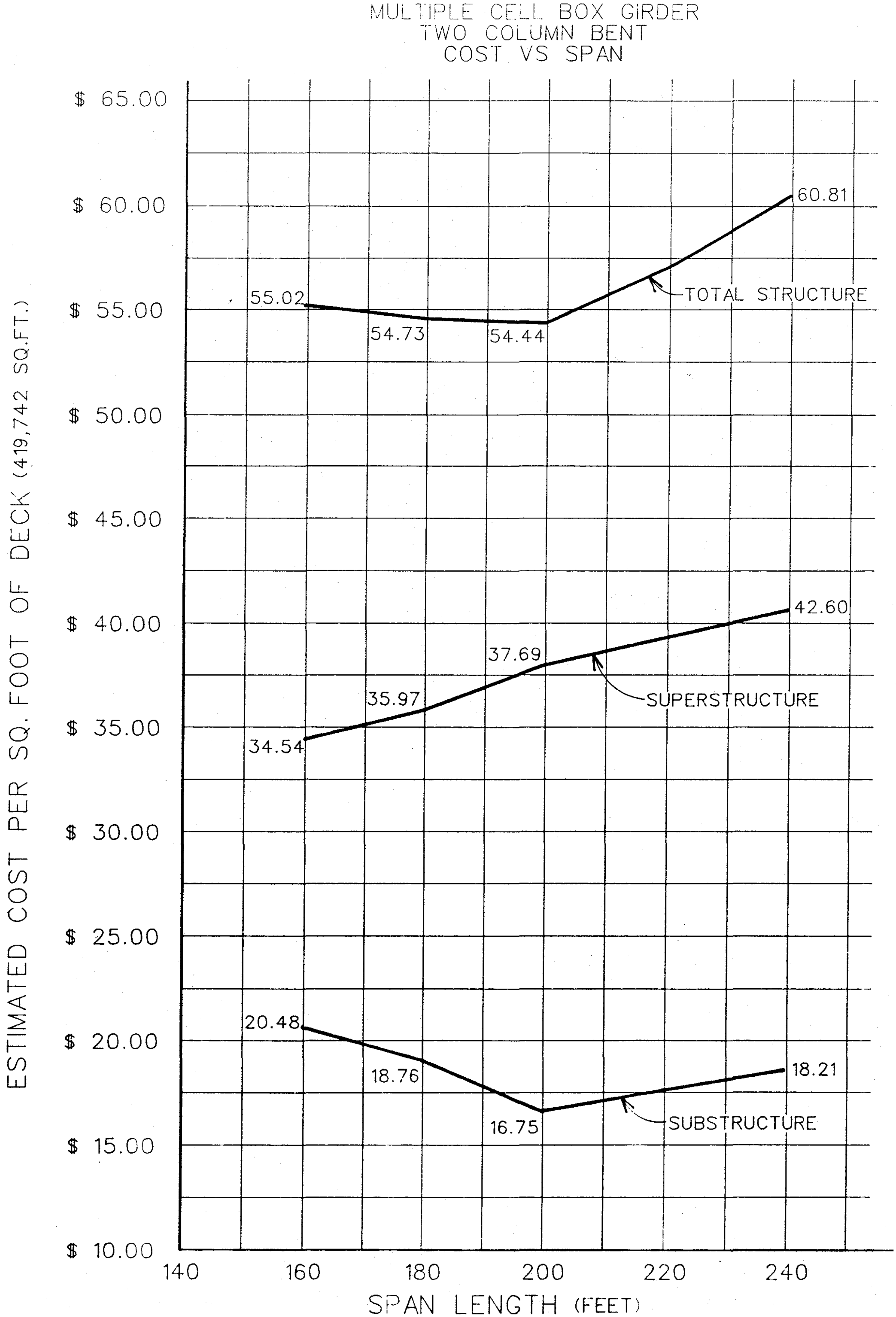

MULTIPLE CELL BOX GIRDER
TWO COLUMN BENT
COST VS SPAN
ESTIMATED COST PER SQ. FOOT OF DECK (419,742 SQ.FT.)
$ 65.00
$ 60.00
$ 55.00
$ 50.00
$ 45.00
$ 40.00
$ 35.00
$ 30.00
$ 25.00
$ 20.00
$ 15.00
$ 10.00
140
160
180
200
220
240
SPAN LENGTH (FEET)
55.02
54.73
54.44
60.81
TOTAL STRUCTURE
34.54
35.97
37.69
42.60
SUPERSTRUCTURE
20.48
18.76
16.75
18.21
SUBSTRUCTURE

FIGURE 9

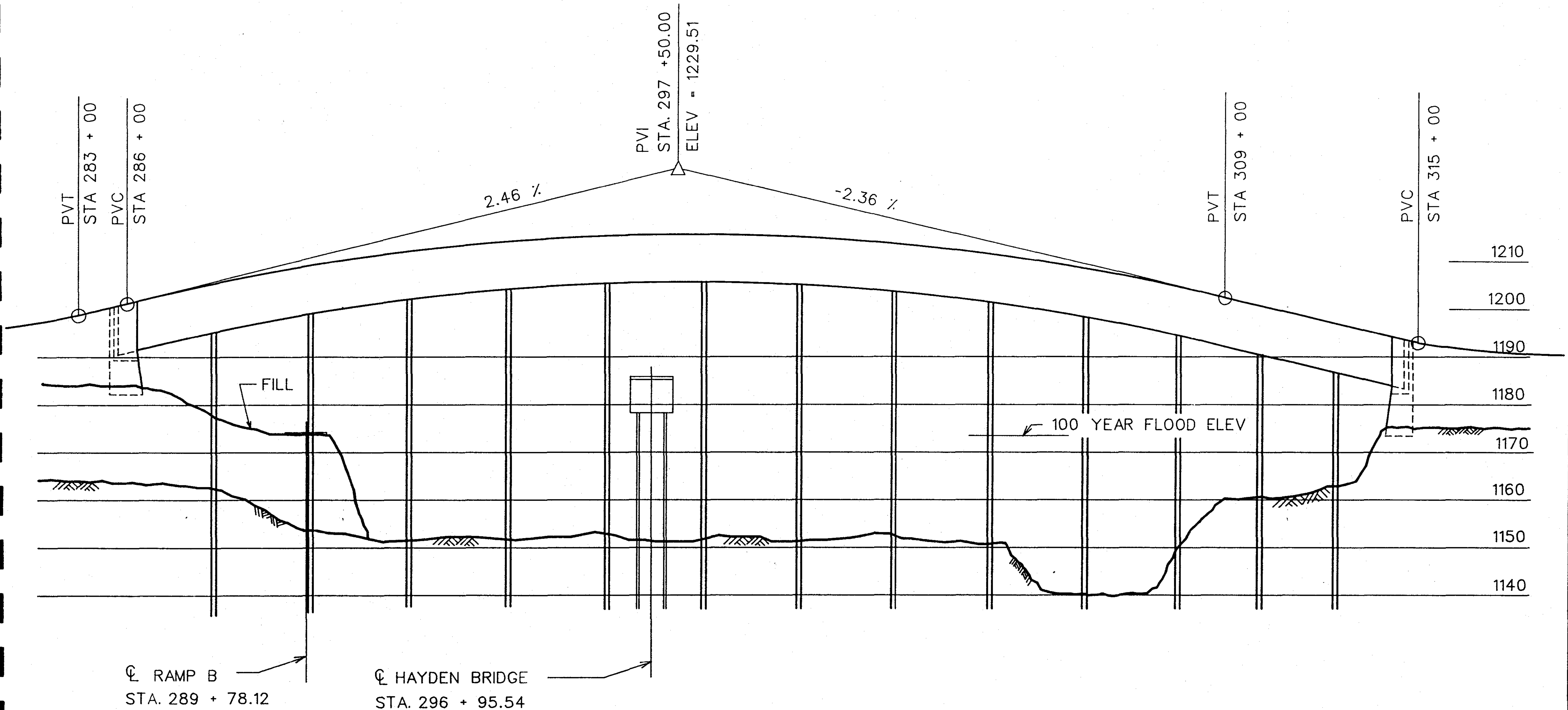
5.2 Bridge Layouts

The span arrangement of the bridges is impacted by constraints of Ramp B, Hayden Road and certain underground utilities. Horizontal clearances to piers adjacent to the off-ramp to Hayden Road (Ramp B) are 15-feet minimum on the left in order to maintain a safe stopping sight distance (400'), and 6-feet on the right with piers behind guardrail or traffic barriers. See Figure 7 for layout adjacent to Ramp B. Piers were located between the Hayden Road Bridge and the underground utilities to avoid the necessity of relocating these utilities. Figure 4 shows the proximity of the piers to the utilities and the Hayden Road Bridge. Discussion of the scour effect is described in Section 4 Drainage Considerations. The west abutment location is controlled by Ramp B. The east abutment is located on the high south bank of the river. Minimum vertical clearance of 16'-6" is provided over Ramp B and Hayden Road. The general span layout for the long span alternates is shown in Figure 6.

A general layout for AASHTO Type VI girders is shown in Figure 5. The controlling span length for this alternate is 145-feet. A comparative estimate made for 127-foot Type VI Modified girder spans indicated no significant difference in cost, therefore, no further consideration was given to this alternate. (Appendix - Section 12). Due to the skew with Ramp B, a longer span cast-in-place box girder segment is required in this area. These units are shown in Figure 7.

Whenever possible, and within the constraints discussed above, the two columns of each pier for both bridges were aligned with the direction of river flow, constraints discussed above. The total length of the eastbound bridge is 2621 feet and the westbound bridge is 2426 feet long.

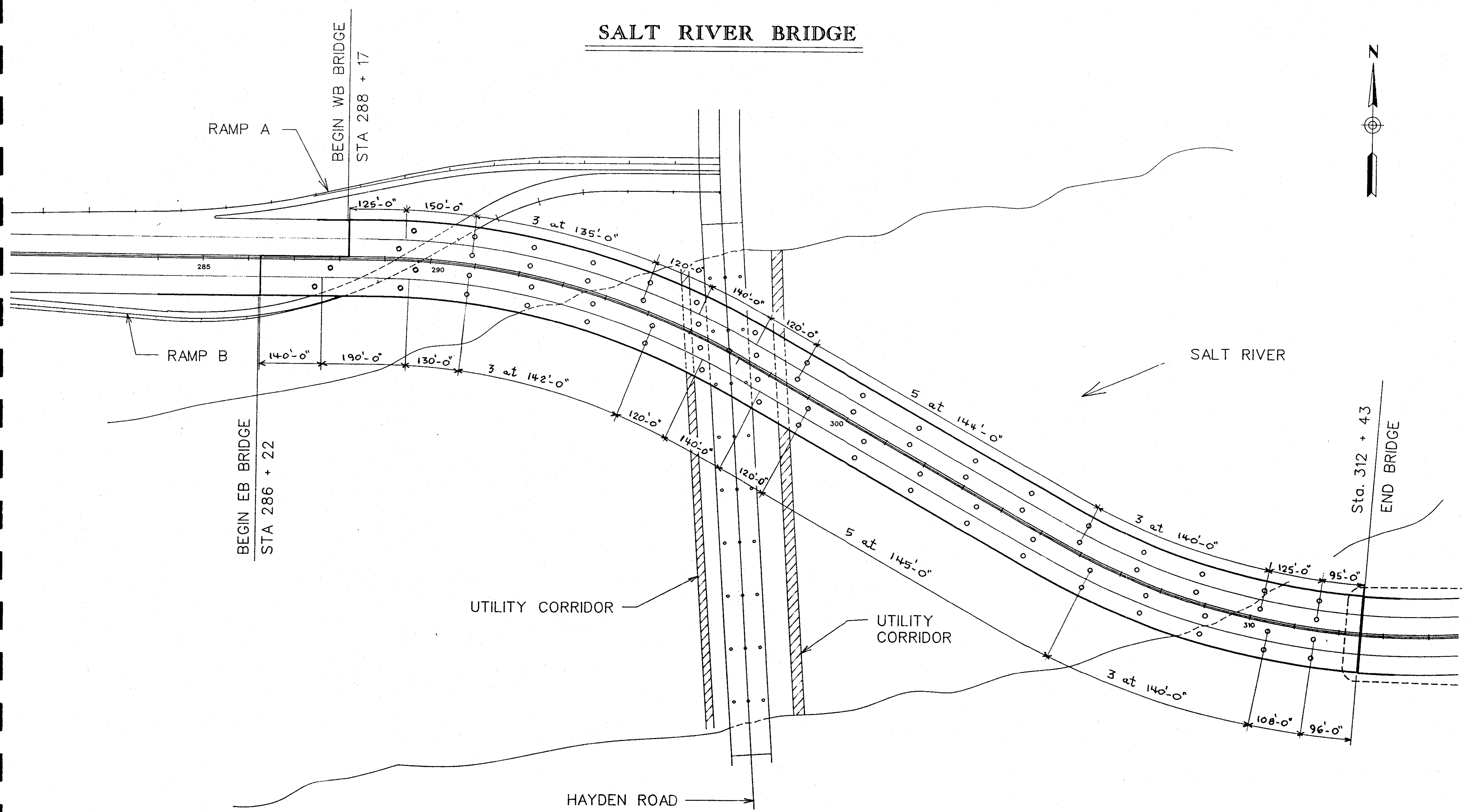
SALT RIVER BRIDGE



PROFILE

FIGURE 3

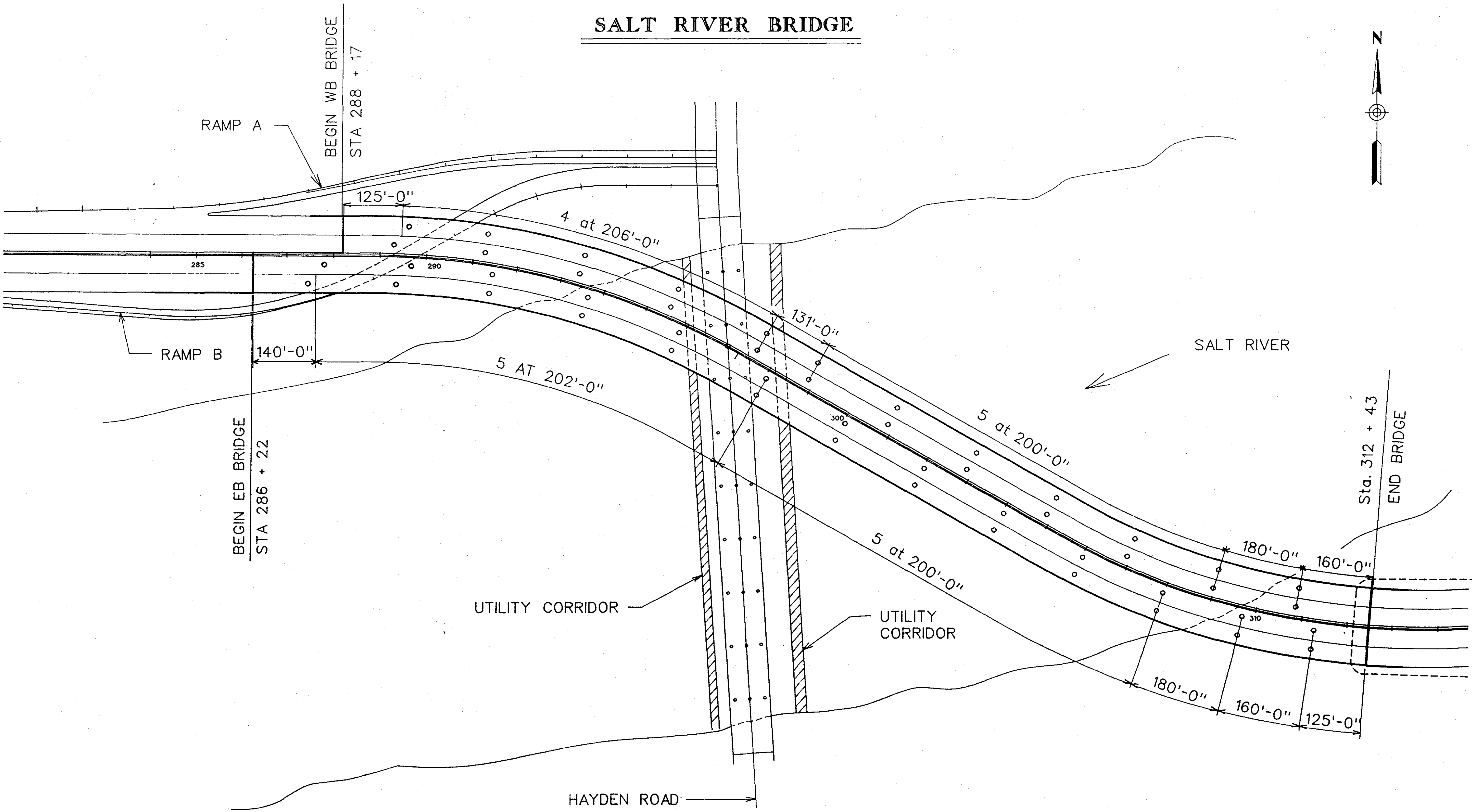
SALT RIVER BRIDGE



145 FT SPANS LAYOUT

FIGURE 5

SALT RIVER BRIDGE



200 FT SPANS LAYOUT

FIGURE 6

SPANS AT RAMP B

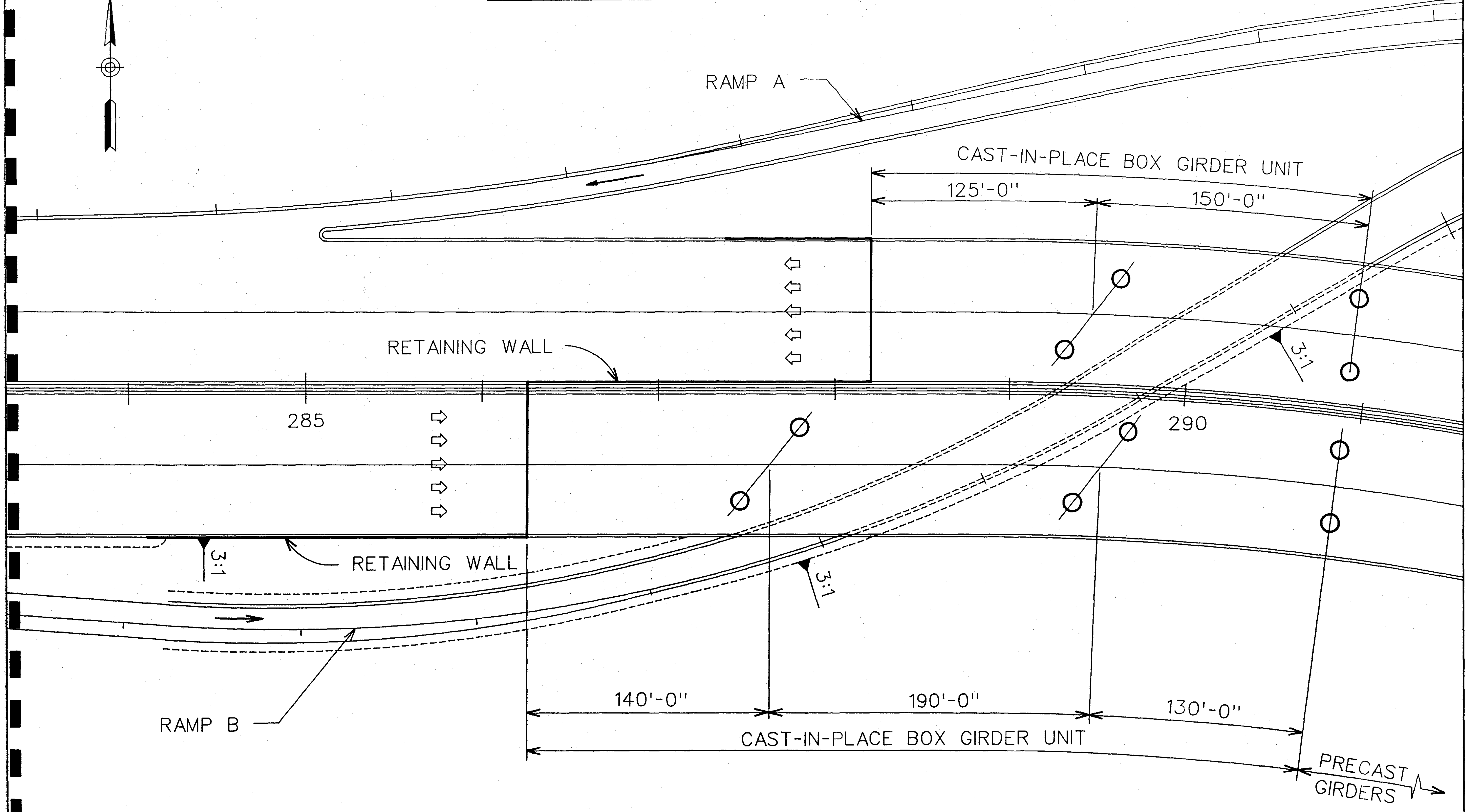


FIGURE 7

WEST ABUTMENT ELEVATION

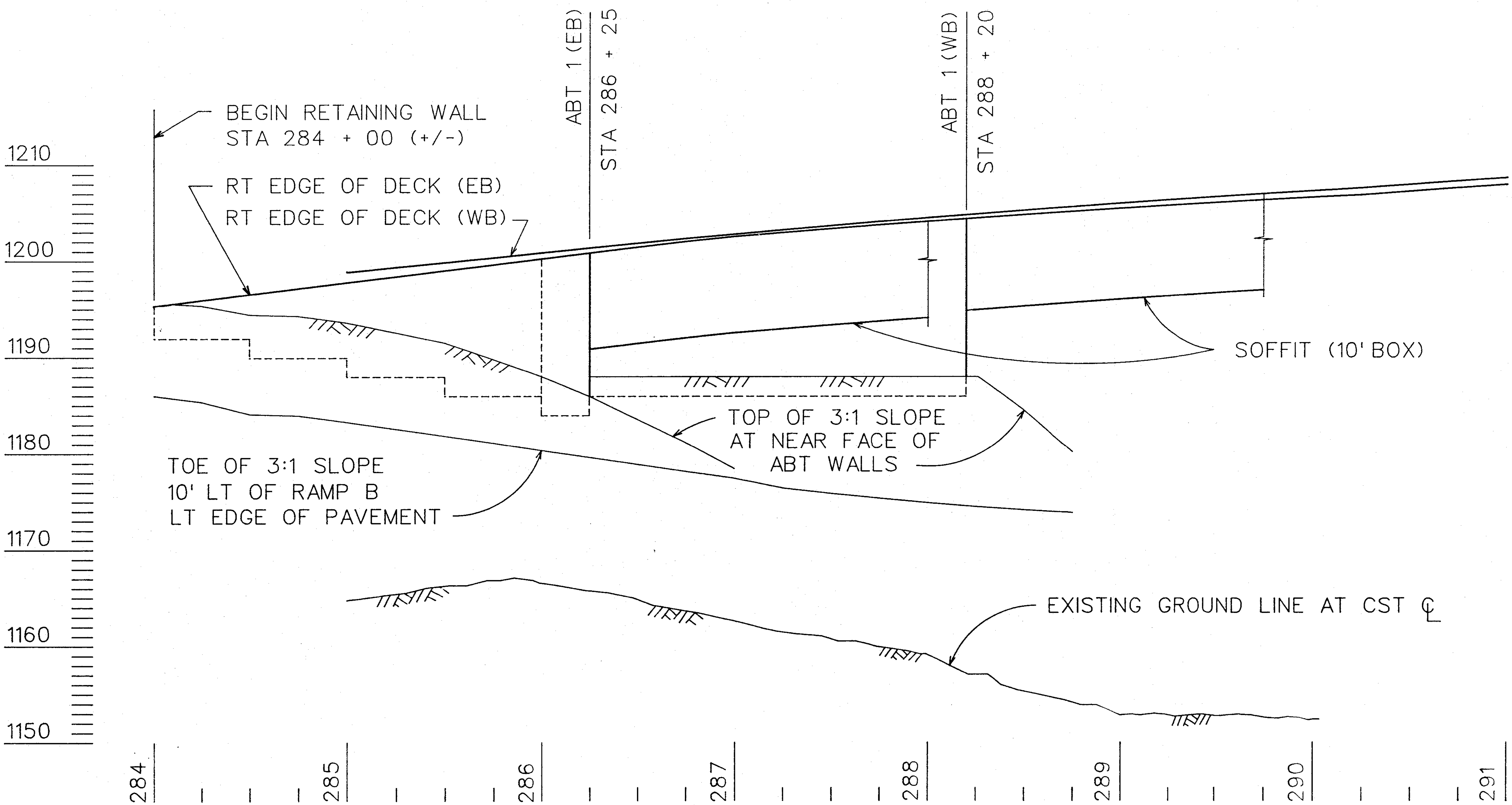


FIGURE 8

6. BRIDGE SUBSTRUCTURE

6.1 Foundations

Two basic types of pier foundations have been considered - single drilled shafts under each column and multiple drilled shafts supporting a cap. Six foundation alternatives were considered:

Two Column Pier

1. Single 10'-0" drilled shaft/column
2. Two 8'-0" drilled shafts with cap/column
3. Four 6'-0" drilled shafts with cap/column

Single Column Pier

4. Single 15'-0" drilled shaft
5. Four 8'-0" drilled shaft with cap
6. Six 6'-0" drilled shafts with cap

Due to the extreme depth of scour potential in the Salt River, the top of the foundation caps would be located 50 feet below stream bed. The cost of excavation and the tremendous volume of concrete necessary for these caps made this type of foundation uneconomical; therefore, further consideration of this type was discontinued.

Poor soil bearing capacity at this site requires long drilled shafts to transmit the load of each column to the soil through friction. (Preliminary geotechnical information is included in the Appendix - Section 12.2). The extreme embedment length of 15-foot diameter shafts required for the single columns precluded the use of the single column piers.

Both 8-foot and 10-foot diameter shafts were estimated for the two column piers. Due to the effective length of the columns in the high scour channel, 10'-0" columns supported by 10-foot diameter shafts are appropriate. Where the effective length of column is less, within the Ramp B embankment, 8'-0" columns and shafts are proposed for economy. Further refinement of column and shaft sizes will be made during final design.

6.2 Piers

The ideal pier configuration would be single column piers for each roadway bridge aligned with the high flow in the river. These piers should be round or another streamline shape and no larger than necessary to minimize scour. This was the original proposal prior to receiving preliminary soils information. Unfortunately, the capacity of the soil to support the high vertical and lateral loads of single drilled shafts for such a wide structure was insufficient within the economic parameters of practical drilling.

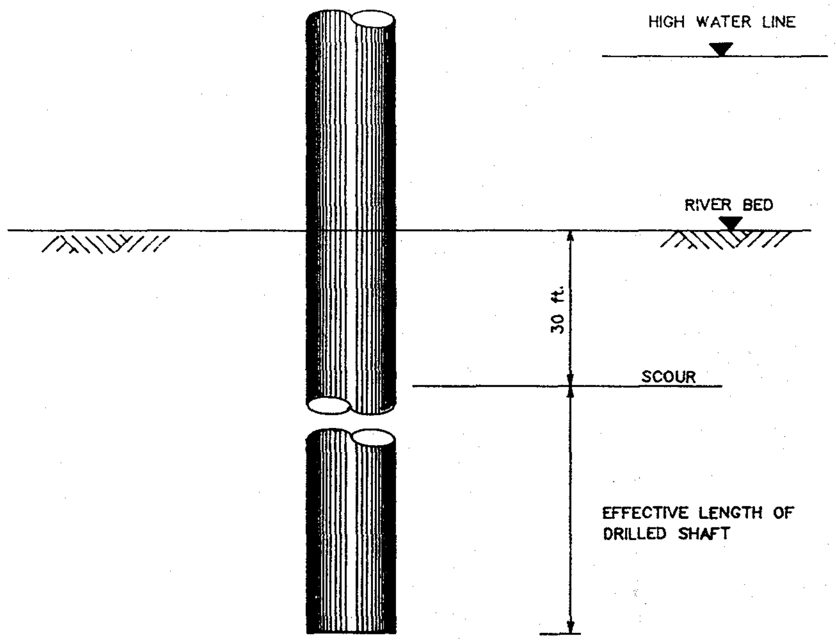
The next best solution was the use of two smaller diameter column piers. These columns can be supported on moderate length drilled shafts 8 to 10 feet in diameter. This size and length shaft is well within the capacity of the drilling contractor industry and should result in good competition.

6.3 Abutments

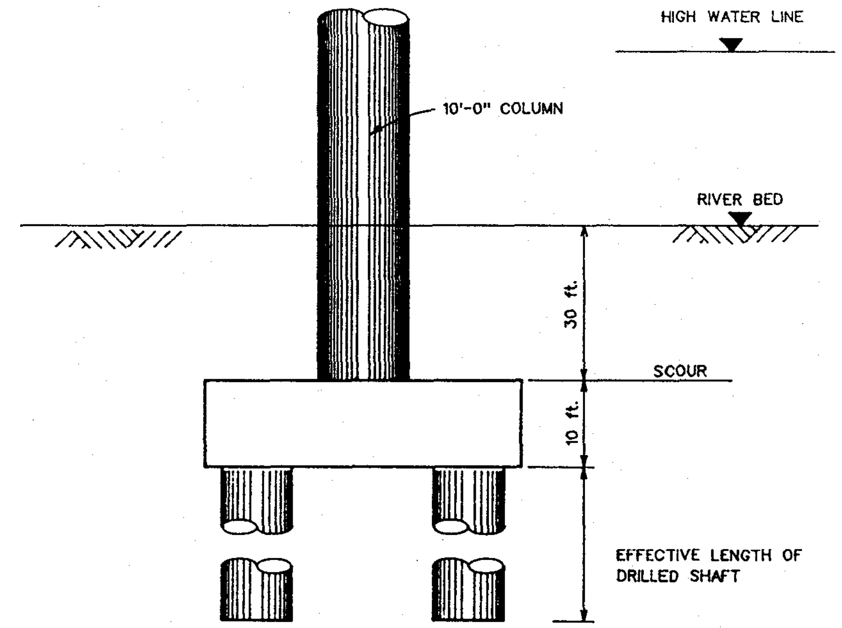
The location of the west abutments was determined by the span arrangement necessary at Ramp B, header bank slope criteria and height of abutment. The proposed location satisfies this criteria and results in moderate height economical abutments as shown on Figures 7 & 8.

The east abutments are minimum height stub type. Both west and east abutments are supported on drilled shafts. A low retaining wall is required along the westbound median shoulder between the west abutments of each roadway. Another retaining wall is necessary along the outside shoulder of the eastbound roadway above Ramp B.

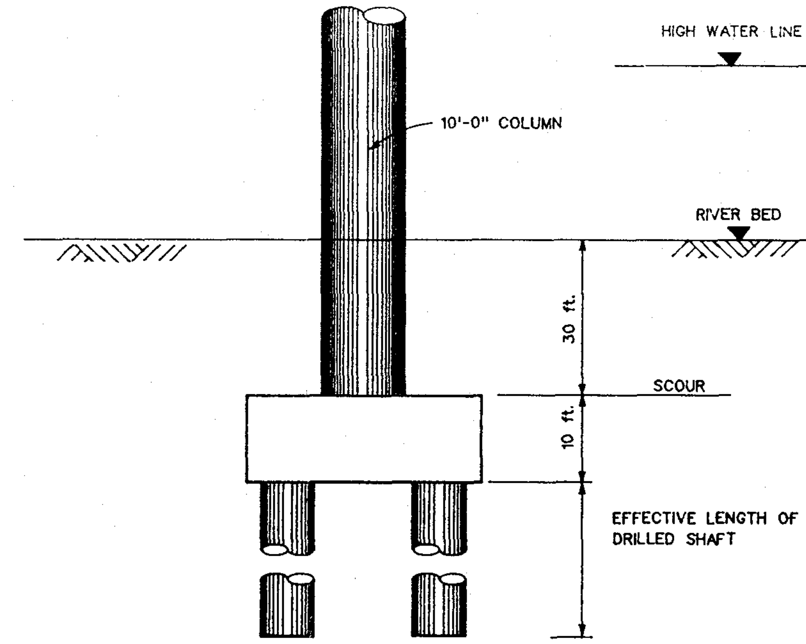
FOUNDATION ALTERNATIVES



SINGLE 10'-0" DRILLED SHAFT

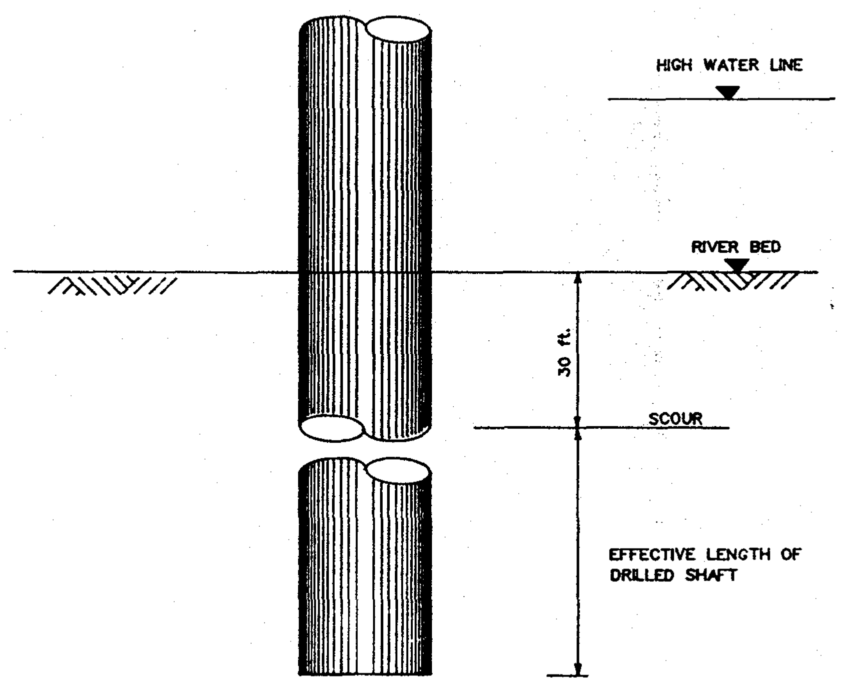


TWO - 8'-0" DRILLED SHAFTS

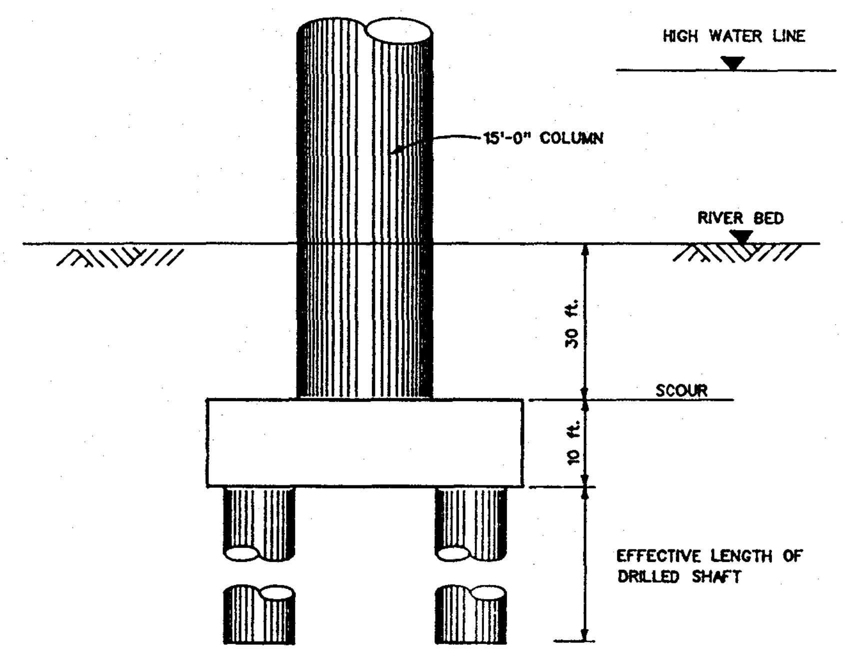


FOUR - 6'-0" DRILLED SHAFTS

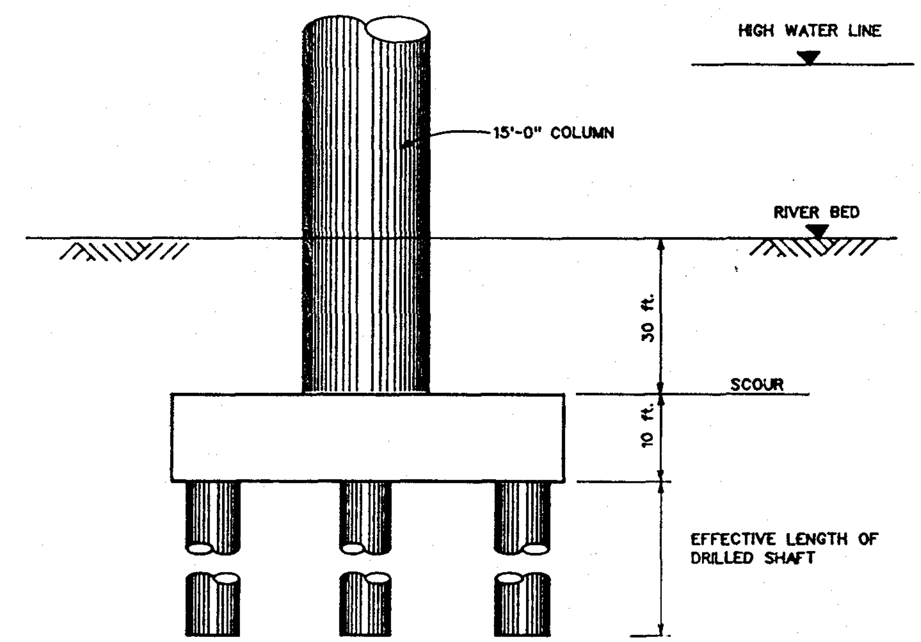
TWO COLUMN BENT



SINGLE 15'-0" DRILLED SHAFT



FOUR - 8'-0" DRILLED SHAFTS



SIX - 6'-0" DRILLED SHAFTS

SINGLE COLUMN BENT

FIGURE 10

7. BRIDGE SUPERSTRUCTURE

From the aesthetic point of view, a bridge of this magnitude and skewed alignment with the river, spanning a major arterial street should be composed of long spans. The long spans would minimize the number of piers in a location already congested with piers at the existing Hayden Road Bridge and provide a spacious overall effect with clean, uncluttered lines and forms. The optimum span length studies described in Section 5 - Span Layout Studies, indicate that span lengths in the 175 to 200 foot range are economical (within the accuracy of the estimates). The typical long span bridge layout shown in Figure 6 was developed and five bridge types suitable for the span lengths considered were selected:

Alternates	Figures	Type
A	11	Cast-in-place multiple cell box girder
B	12	Cast-in-place twin box girder
C	Sec. 12.5	Precast AASHTO Type VI girder (drop-in-span)
D	Sec. 12.5	Precast trapezoidal box girder (drop-in-span)
E	14	Curved steel plate girders

A sixth Alternate (F) consisting of 145-foot AASHTO Type VI girder spans was added later. (Figure 13)

Comparison of Alternates A and B during preliminary design indicated that the twin box girder required less concrete and therefore less foundation capacity and had significant construction advantages over the single multi-cell box girder. Accordingly Alternate A was eliminated from further consideration.

The precast drop-in Alternates (C&D) were included to provide a bridge that could be erected with minimum falsework in the river. However, questions were raised regarding the number of expansion joints and rideability of such a structure, therefore these Alternates were also eliminated.

7.1 Twin Post-tensioned Box Girders

The cross section shown in Figure 11 consists of two 41'-0" wide boxes separated by a 1'-2" closure strip. The two boxes are joined by a diaphragm at the pier, which gives frame action to the bent. The 3 and 4 span structural units are monolithic with the interior piers with expansion joints and bearings located at the exterior piers.

The box girders would be constructed on falsework (if permitted) or on a launching truss as shown in Figure 15 or some combination of the two.

7.2 Curved Steel Plate Girder

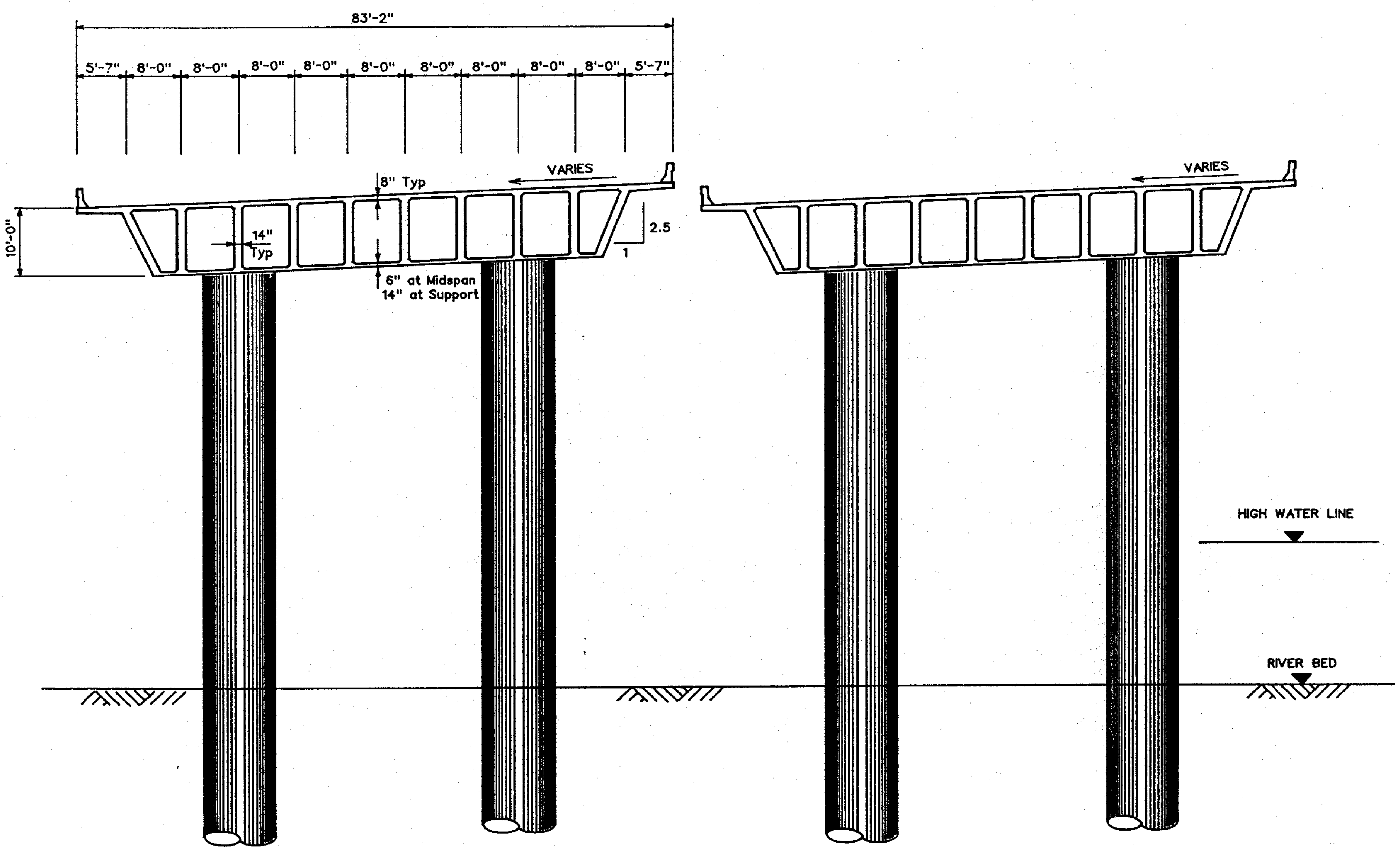
The cross section shown in Figure 14 consists of six lines of curved structural steel plate girders spaced at 14'-0" on center supporting a 9" composite deck. Transverse stiffners on the exterior girders will be at cross frames only. No longitudinal stiffeners are required. Primary members consist of A572 structural steel with A36 secondary members. Shop splices would be welded with field splices bolted. All structural steel would be shop primed and field painted.

The superstructure is supported by two-column bents on drilled shafts.

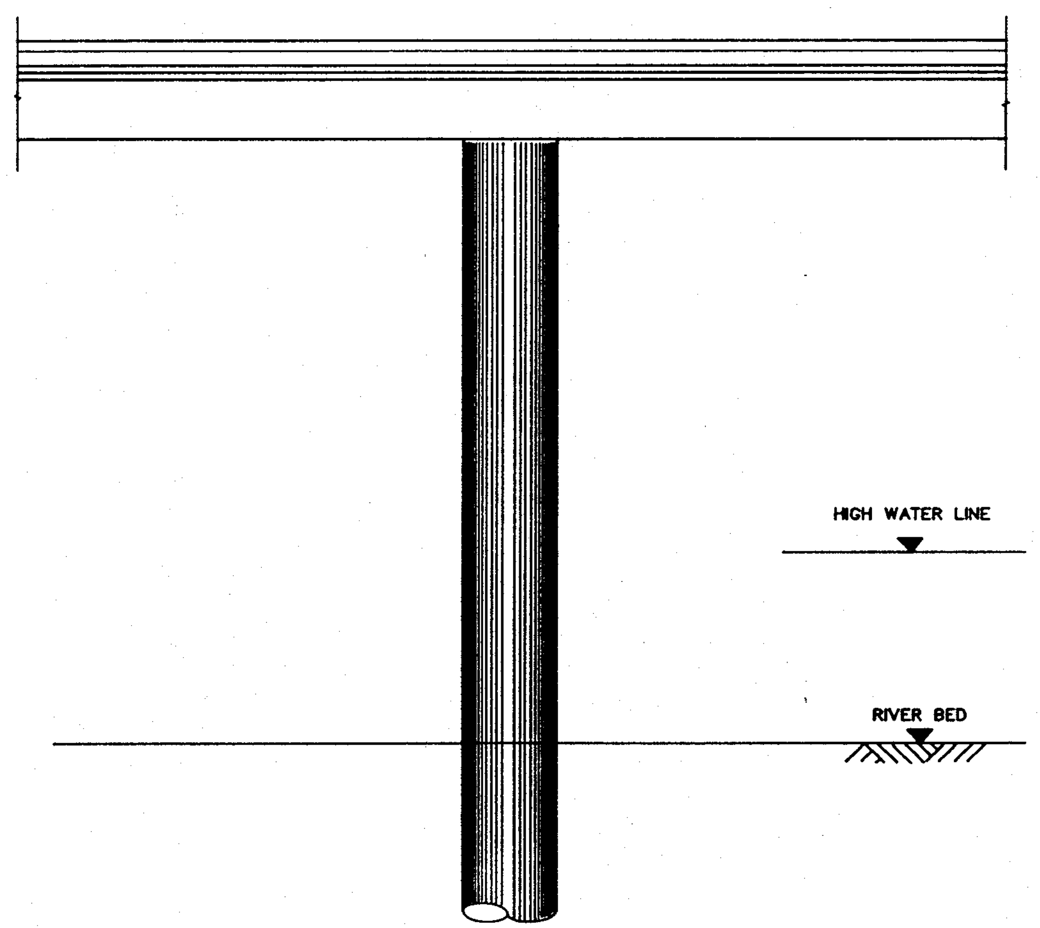
7.3 AASHTO Type VI Girders

The cross section for the 145-foot span length alternate shown in Figure 13 consists of fourteen lines of Type VI Girders on 6'-0" centers supporting a 7 1/2" composite deck. The layout consists of 3 and 4 span structural units. Due to the extreme skew of Ramp B, the length of span necessary at this location exceeds the limit of precast girders. A cast-in-place box girder unit is proposed for the section from the west abutment to just east of Ramp B. The depth of this unit will be designed to blend to the precast adjacent section without an abrupt transition.

CIP MULTIPLE CELL BOX GIRDER



TYPICAL SECTION

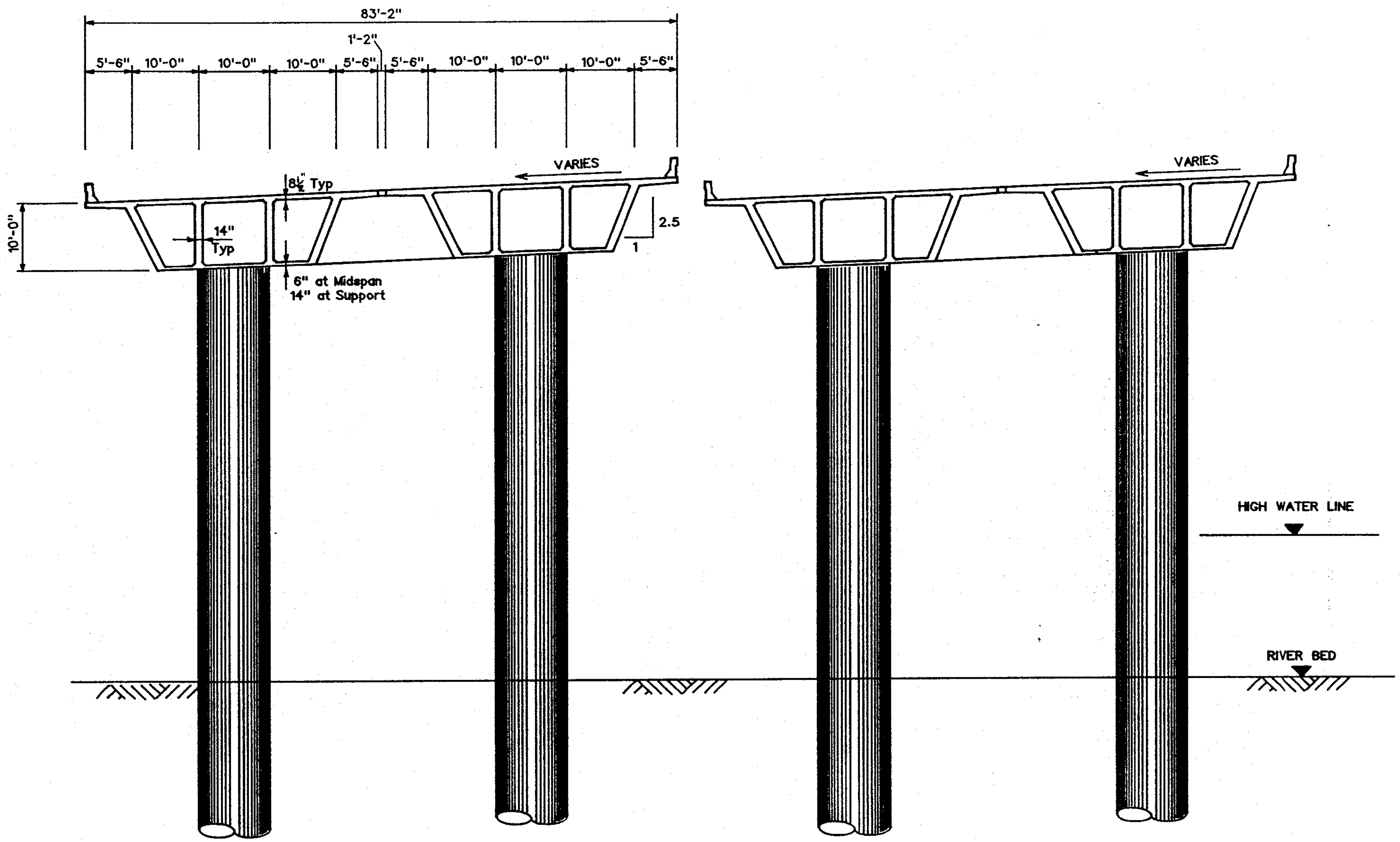


ELEVATION AT PIER

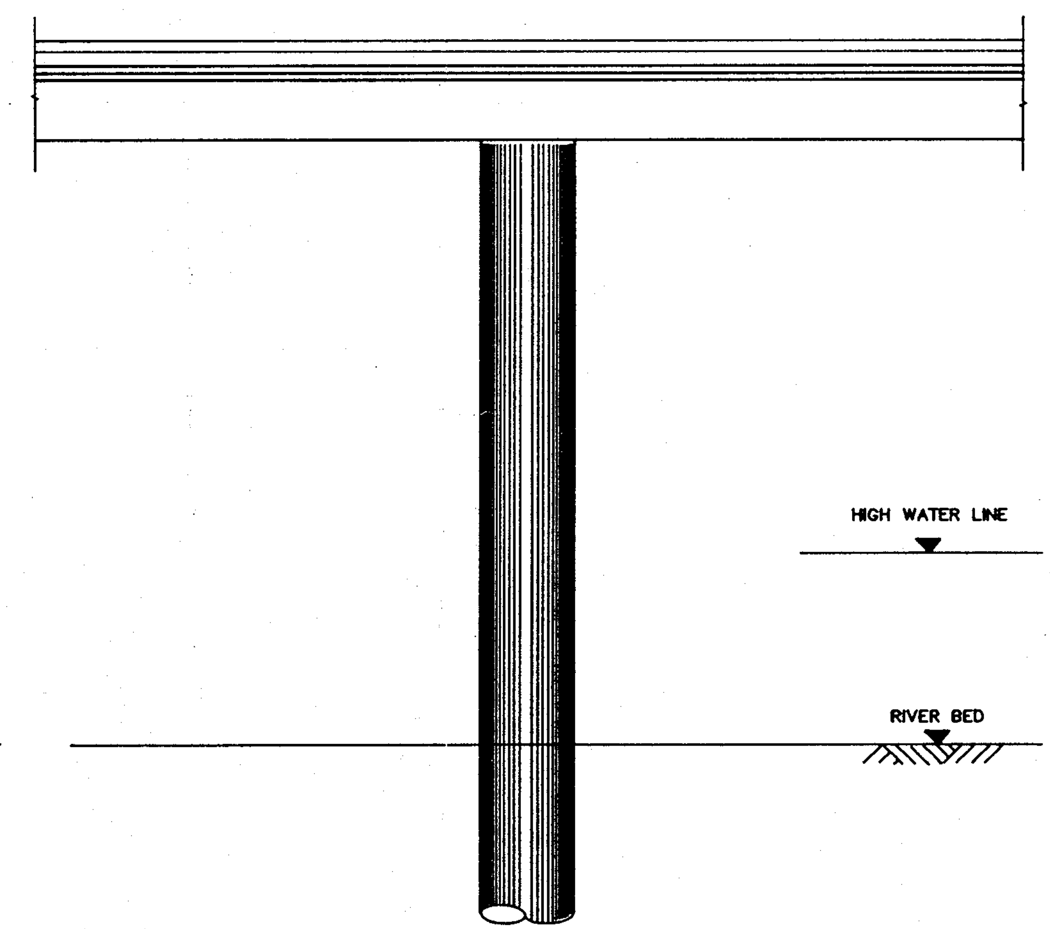
TWO COLUMN BENT

FIGURE 11

CIP TWIN BOX GIRDER



TYPICAL SECTION

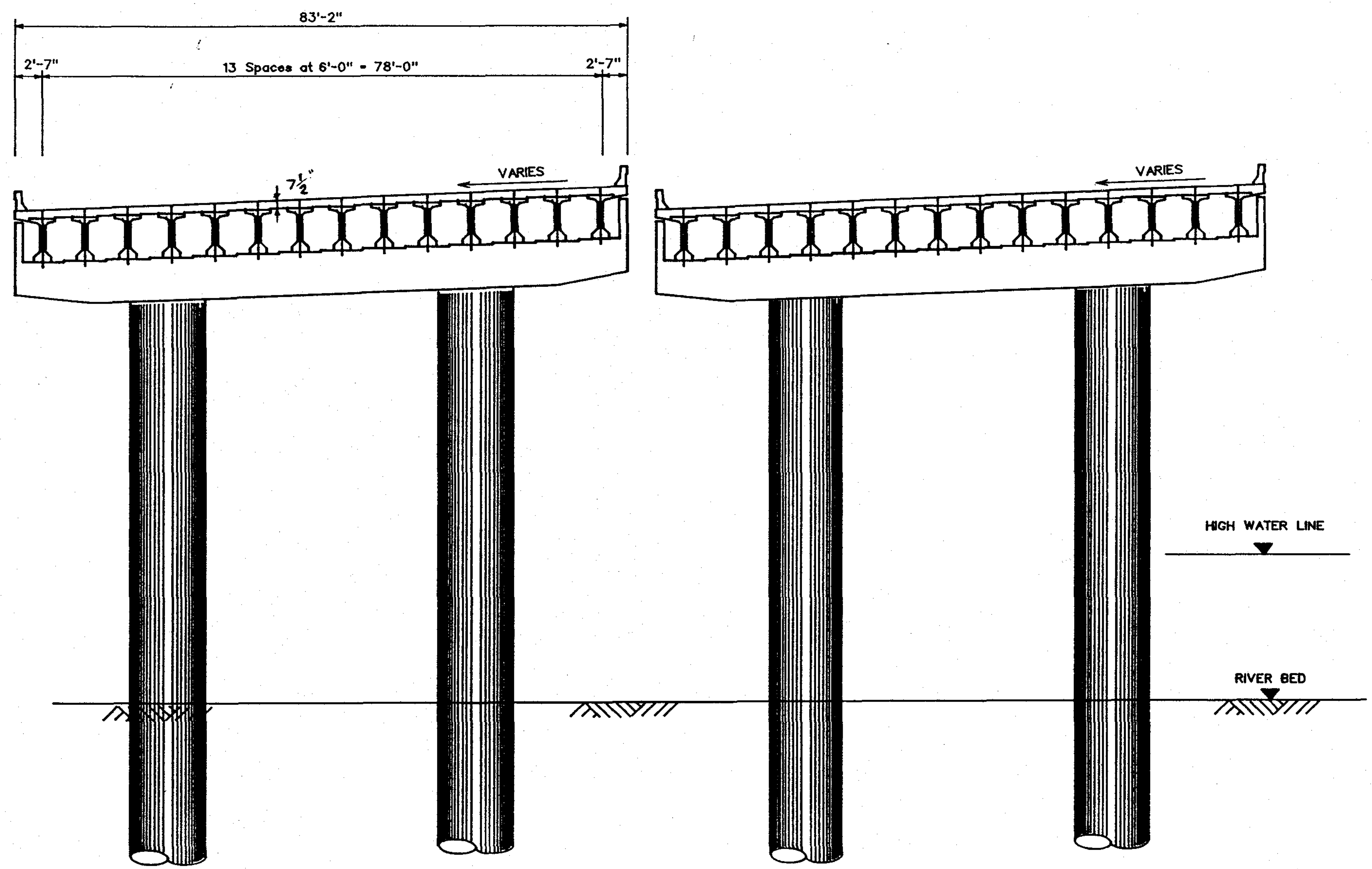


ELEVATION AT PIER

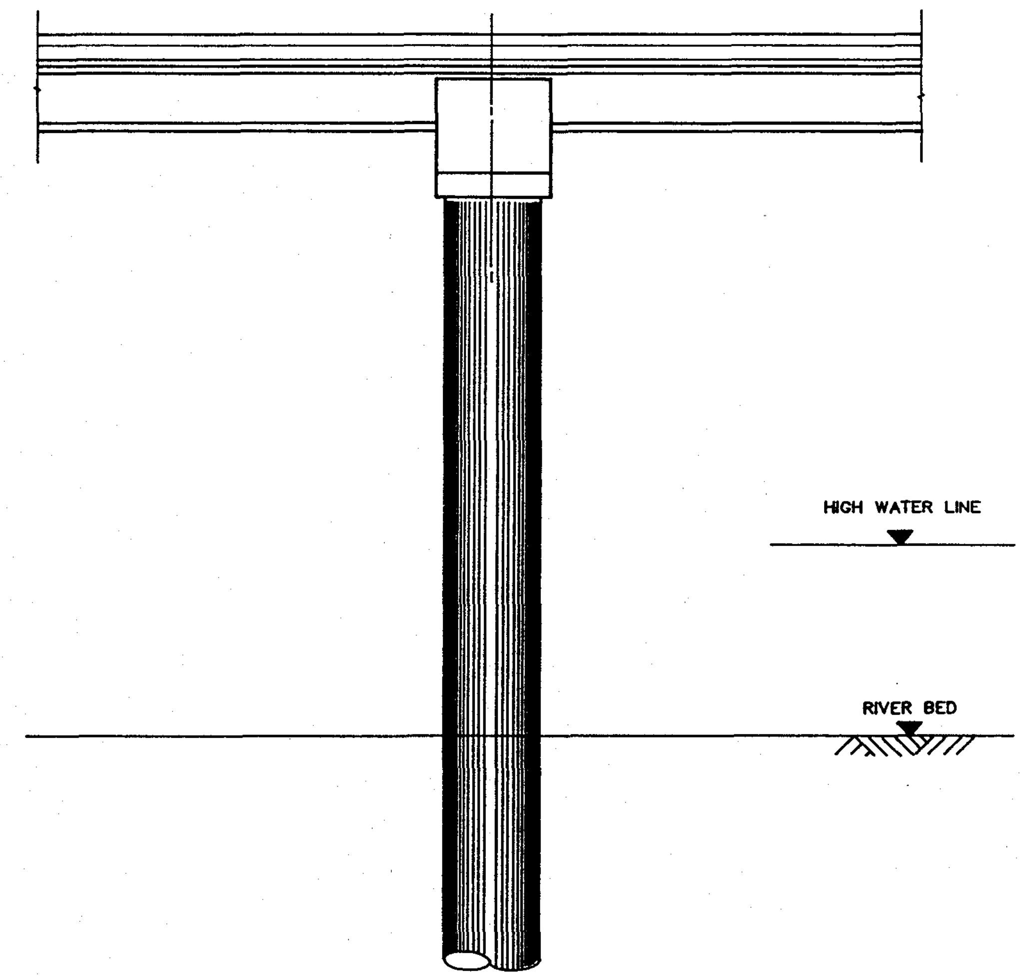
TWO COLUMN BENT

FIGURE 12

AASHTO VI (145 FT SPAN)



TYPICAL SECTION

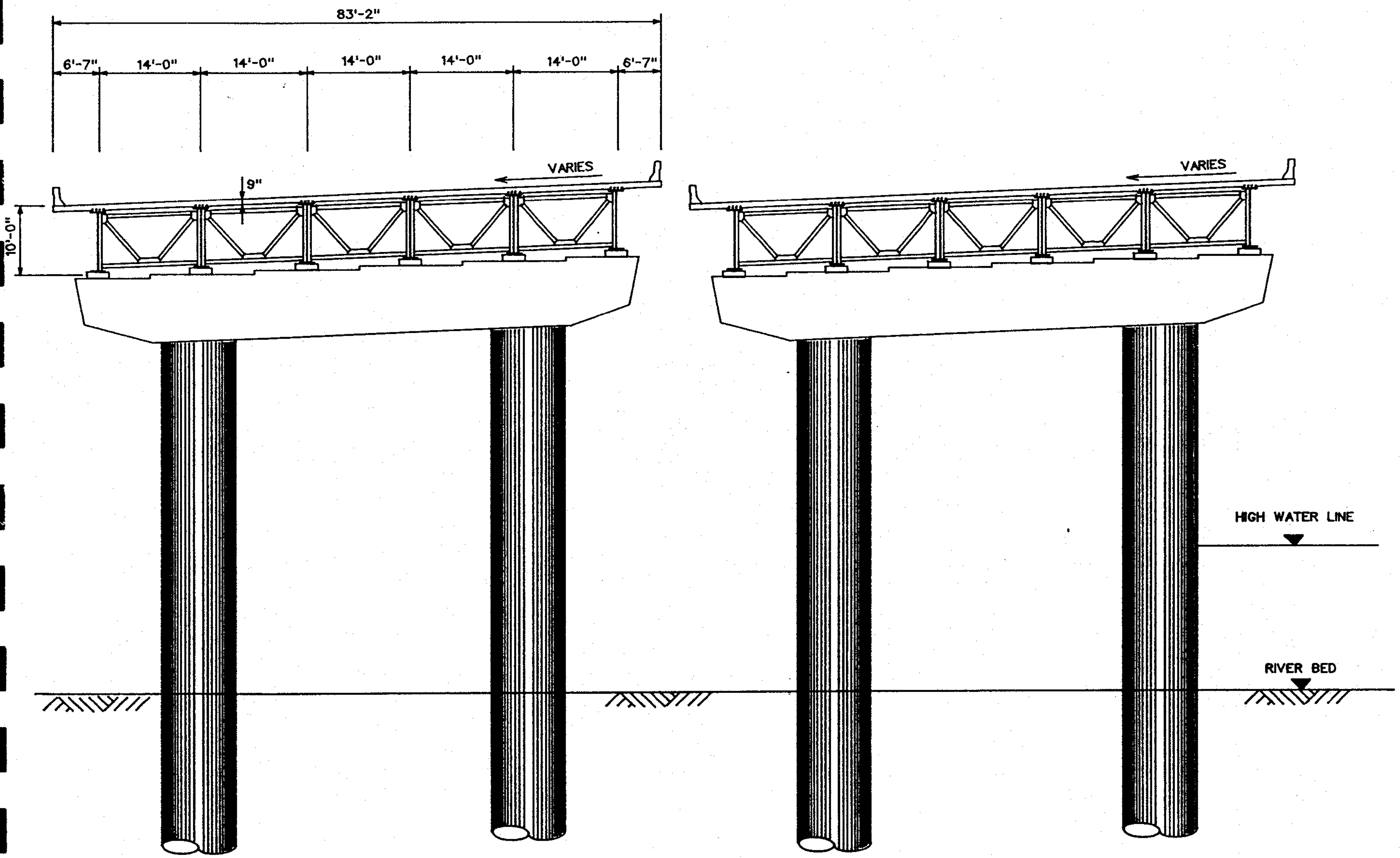


ELEVATION AT PIER

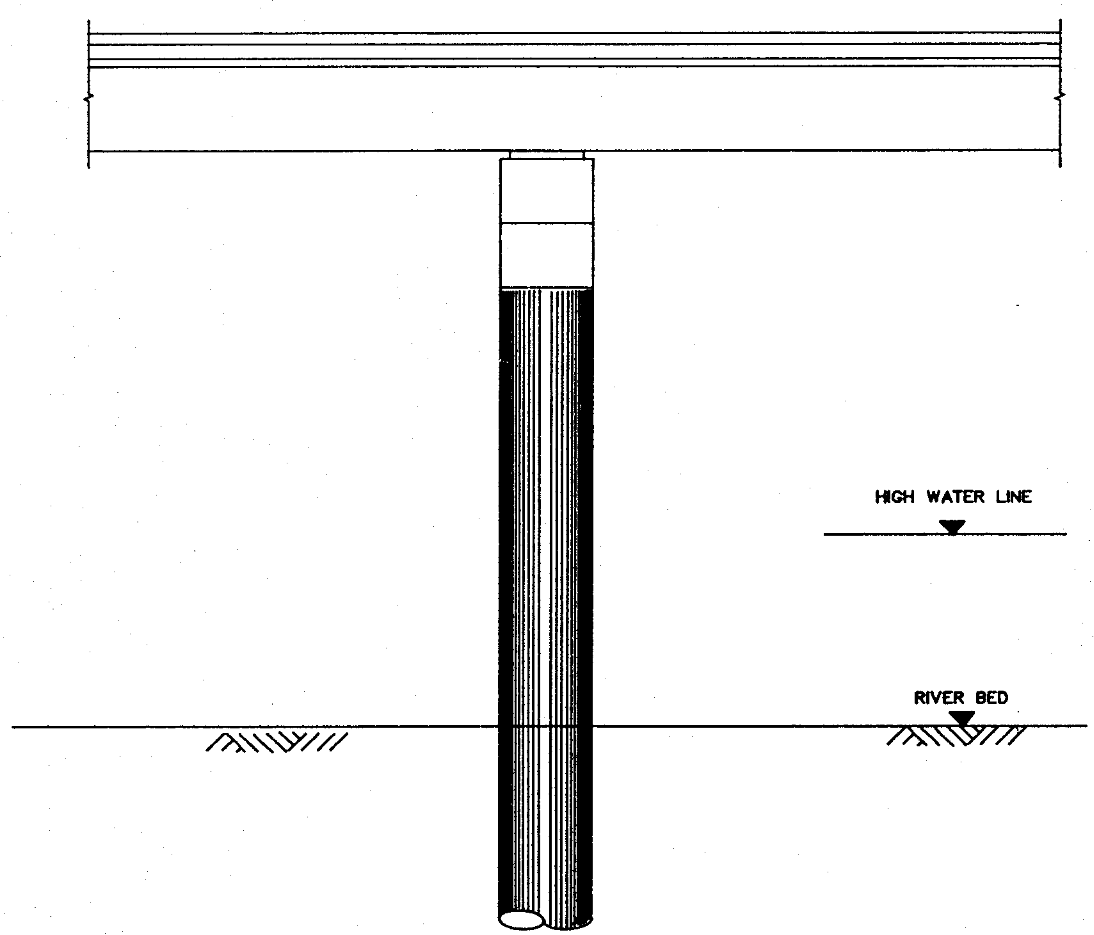
TWO COLUMN BENT

FIGURE 13

STEEL PLATE GIRDER



TYPICAL SECTION



ELEVATION AT PIER

TWO COLUMN BENT

FIGURE 14

8. CONSTRUCTION PROCEDURE

8.1 Substructure

Two column piers are proposed for all of the alternates. The column sizes will vary between 8' and 10' and can be constructed in conventional steel forms with form liners to produce the rustications. Forms for the cap beams for the precast girder and steel alternates can be supported from the columns. The single drilled shaft under each column will vary in size from 8' to 10' in diameter. These size shafts have been used in the Phoenix area and can be auger drilled by any of the major foundation contractors. Partial length casings will probably be necessary. The shaft lengths required will penetrate the water table. Slurry drilling may be needed in some holes. The abutments will be cast-in-place on fill and supported by an array of 4-foot diameter drilled shafts.

8.2 Superstructure

8.2.1 Cast-in-Place Girders (Alternate B)

Construction of cast-in-place box girders would be supported by falsework or a launching truss. Falsework has the advantage of being more economical, but there is some risk of destruction due to flooding. Fortunately the extreme high flows in the Salt River are infrequent and seasonal. We believe consideration should be given to permitting falsework under restrictive conditions written into the Special Provisions. A list of spills from Granite Reef Dam and major drains upstream from this site are included in the Appendix - Section 12.3. This data, together with other hydrologic information, could be used to develop specifications controlling the amount of falsework allowed in the river at any time. The post-tensioning design would be tailored to fit the construction sequence desired. This could vary from span-by-span type construction to 3 or 4-span continuous construction.

One alternative to building on falsework is the use of a launching truss such as that shown and described in Figure 15. The launching truss consists of two longitudinal trusses connected by a transverse floor system which supports the formwork. The truss must be at least two spans long since portions of two spans are cast at the same time. In this case it must have sufficient articulation to accomodate the 3°-30' curves. In order to reduce the size of the truss members, a temporary support consisting of a strut bearing on a precast pad could be placed at midspan or a king post truss may also be utilized. In order to complete the project on schedule, two separate launching truss units will be required. The estimated cost of these truss units is $2 million and is included in the cost summary for Alternate B.

Segmental construction could also be considered as a viable construction option at this site. The span lengths are at the low end of economical balanced cantilever erection but the span-by-span method may be competitive. Further discussion of segmental construction is included in the Appendix - Section 12.5.

8.2.2 Precast Girders (Alternate F)

The 145' spans for the precast girder alternate are the maximum bridge span length for AASHTO Type VI girders. These girders would be erected by two cranes from the ground. The layout consists of 3 and 4-span units with girders bearing on elastomeric pads. The first unit, beginning at the west abutment and spanning Ramp B, consists of cast-in-place box girders due to the longer spans necessary. Falsework for these spans would be constructed from Ramp B embankment and would not be subject to flooding.

8.2.3 Structural Steel Plate Girders (Alternate E)

The curved plate girders would be erected by cranes from the ground. Field splices would be made with high strength bolts. The composite concrete deck would be finished with conventional bridge deck finishing machines supported by screed rails located on outrigger forms.

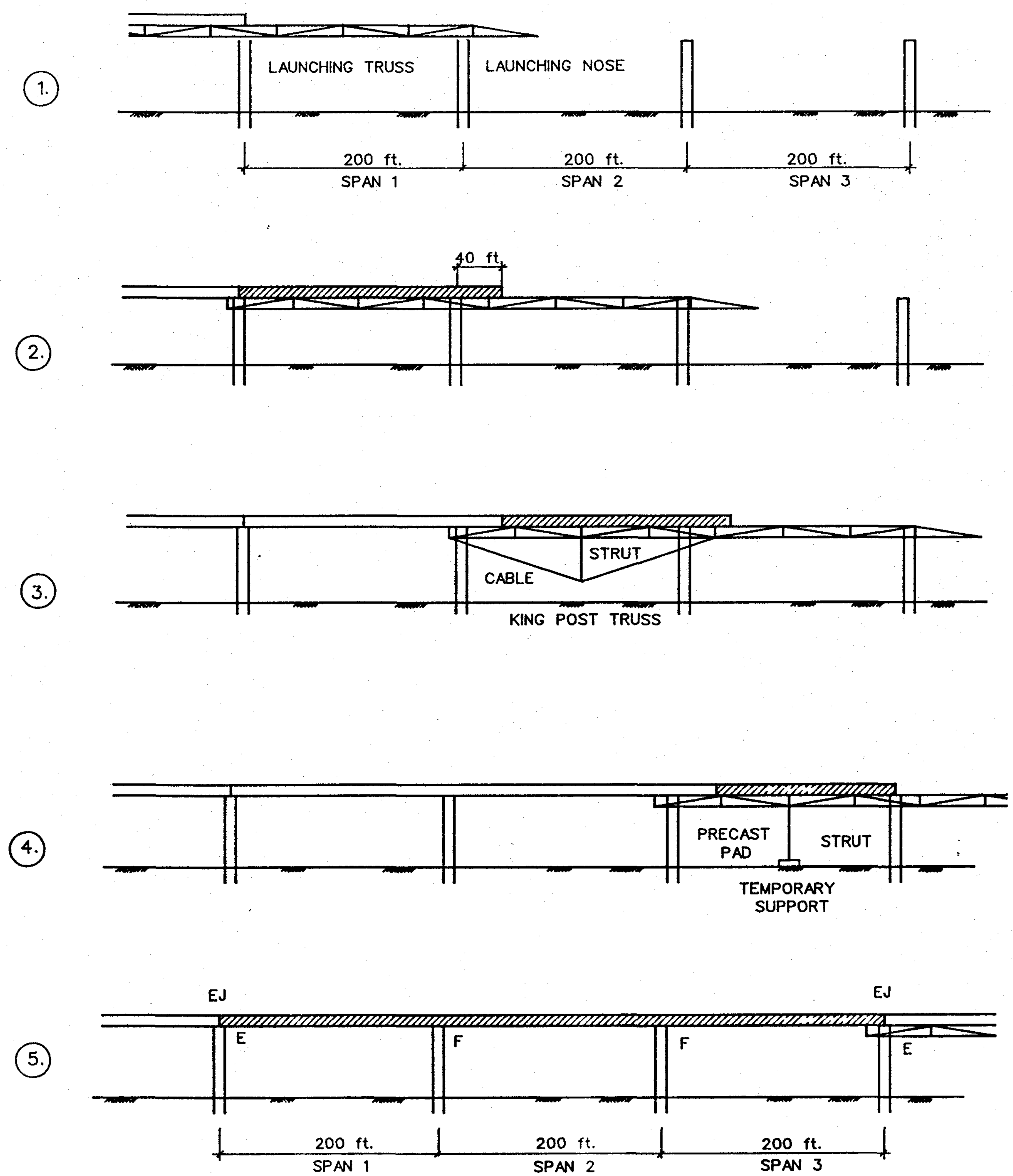
8.3 <u>Construction Clearances</u>

The 230 KV transmission lines parallel to Hayden Road must be raised to provide construction clearances for the cranes and tall drilling machines. The 69 KV and smaller lines must be temporarily rerouted for construction.

Construction of the cast-in-place girders on falsework over Hayden Road will require a temporary

bent on the center line of Hayden Road which will result in some disruption to traffic. Erection of either precast or steel girders over Hayden Road can be during off-peak hours with only short temporary disruption to traffic.

CONSTRUCTION SEQUENCE - TYPICAL 3 SPAN STRUCTURAL UNIT



ELEVATION

1. ADVANCE LAUNCHING TRUSS TO SPAN 1.

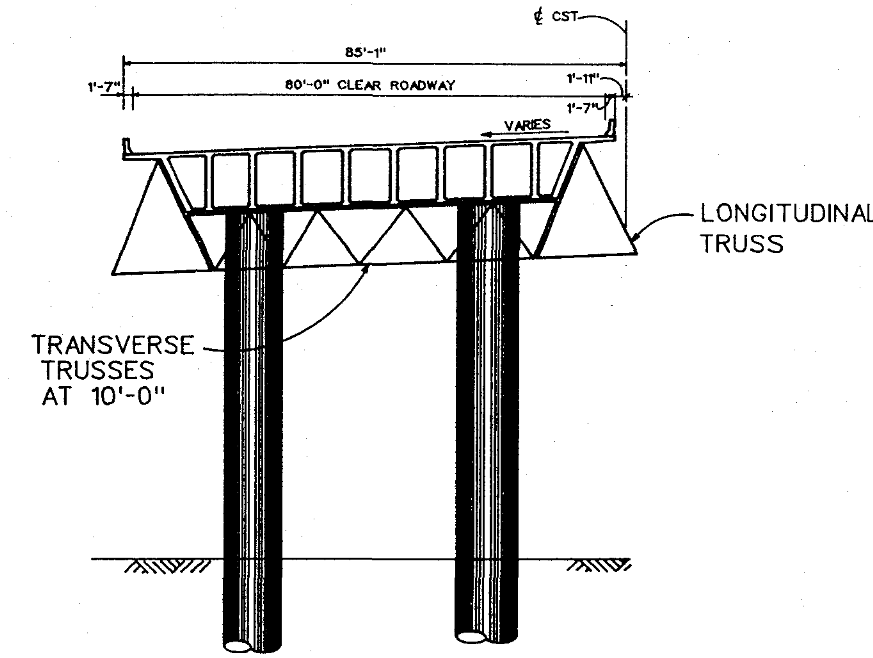
2. ADVANCE LAUNCHING TRUSS TO SPAN 2.
CAST SPAN 1 UP TO INFLECTION POINT OF SPAN 2.
USE COUPLERS TO ATTACH POST - TENSIONING
TO SUBSEQUENT SPANS.

3. ADVANCE LAUNCHING TRUSS TO SPAN 3.
CAST SPAN 2 UP TO INFLECTION POINT OF SPAN 3.
KING POST TRUSS OR TEMPORARY SUPPORT
(SHOWN BELOW) SUPPORTS WEIGHT OF WET CONCRETE.
LAUNCHING TRUSS SUPPORTS OWN WEIGHT DURING
LAUNCHING.

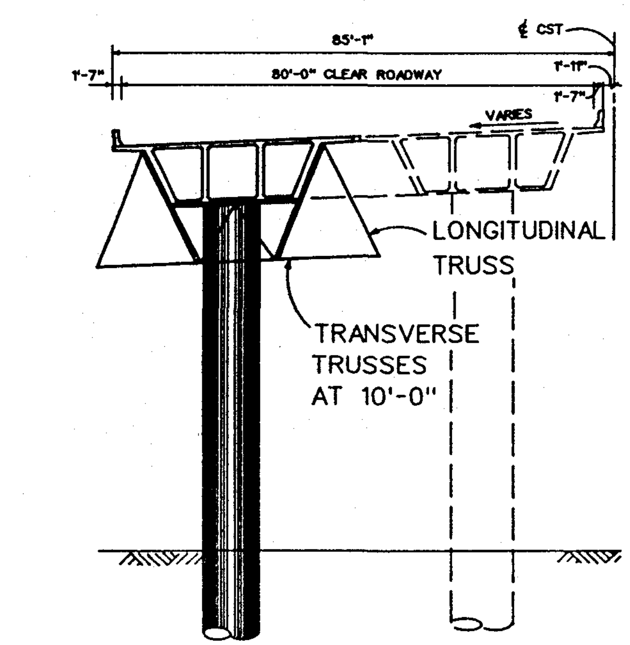
4. ADVANCE LAUNCHING TRUSS TO NEXT STRUCTURAL UNIT.
CAST SPAN 3.

5. COMPLETED 3 SPAN STRUCTURAL UNIT.

LEGEND :
EJ - EXPANSION JOINT
E - EXPANSION BEARINGS
F - FIXED SUPPORT



MULTIPLE CELL BOX GIRDER



TWIN BOX GIRDER

FIGURE 15

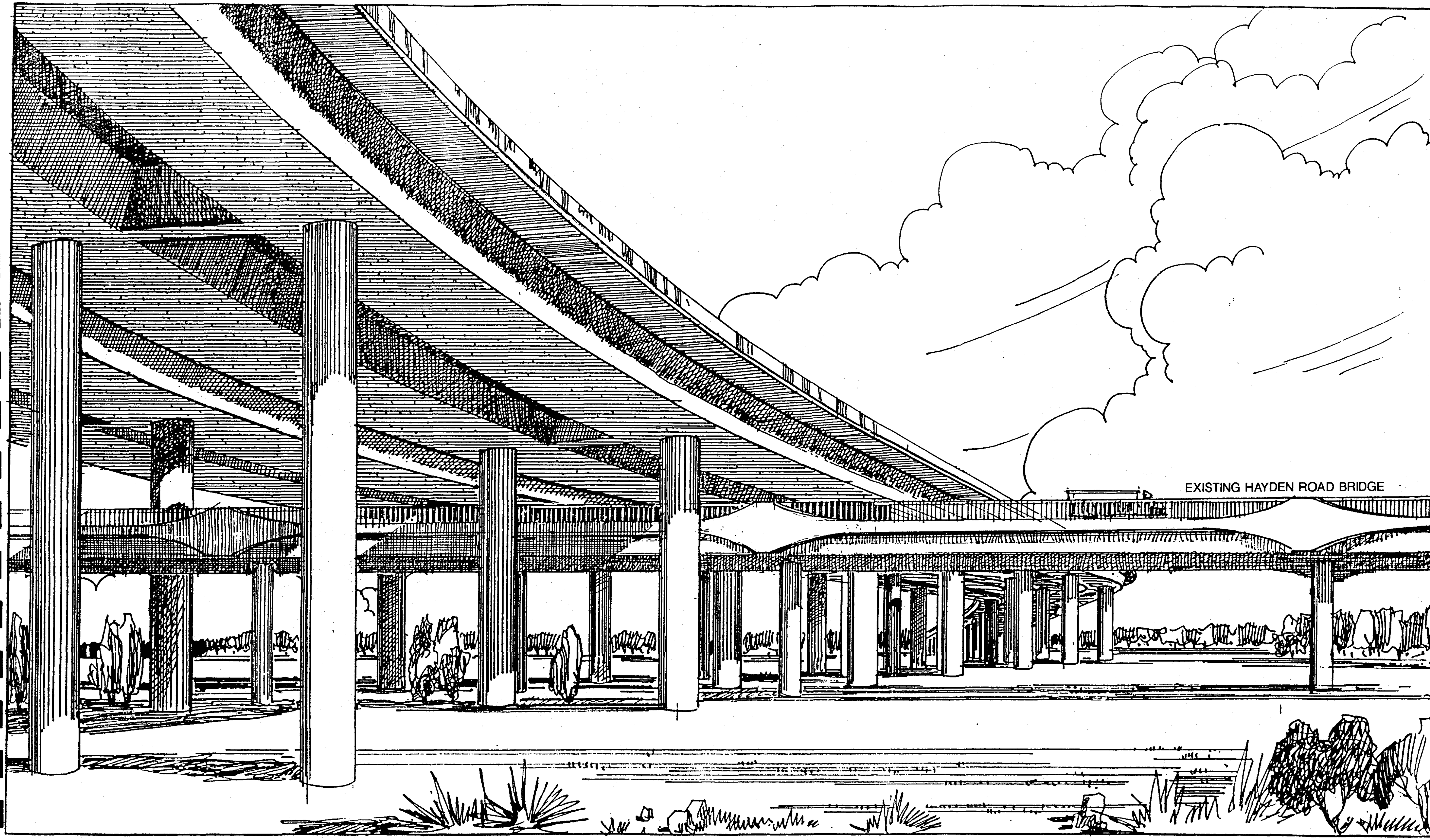
9. AESTHETICS

9.1 General Appearance

Long spans with few piers tend to produce a more attractive bridge. This is particularly evident at this site with the skewed alignment to the river and superelevated reverse curves. Shorter spans would create a less attractive cluster of piers. The twin concrete box girder alternate appears to best satisfy the aesthetic criteria. The superstructure ratio of depth to the span length, height of structure and column size results in pleasing proportions. The smooth bottom slab of this type of bridge is also preferrable to multiple girders and will be very noticeable due to the superelevation.

9.2 Architectural Treatment

Rustications 3" wide by 1 1/2" deep are suggested for the piers, walls and abutments of all of the alternates. This rustication pattern is consistant with that recently adopted for the East Papago Freeway piers. The ADOT standard Type A barrier would also provide some visual relief.



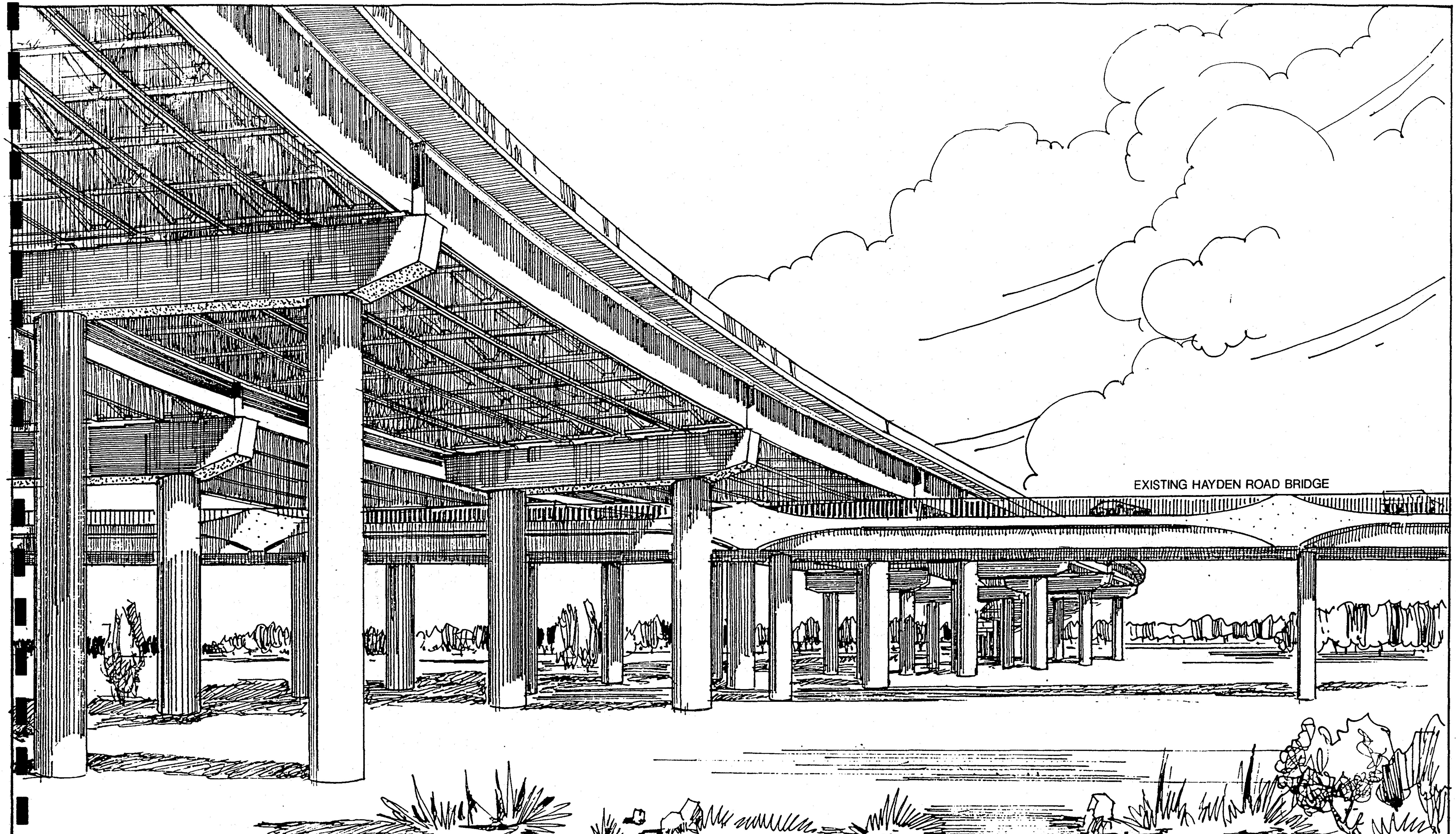
SALT RIVER BRIDGE

EAST PAPAGO FREEWAY

POST-TENSIONED CONCRETE TWIN BOX GIRDERS

200-FOOT SPANS

FIGURE 16



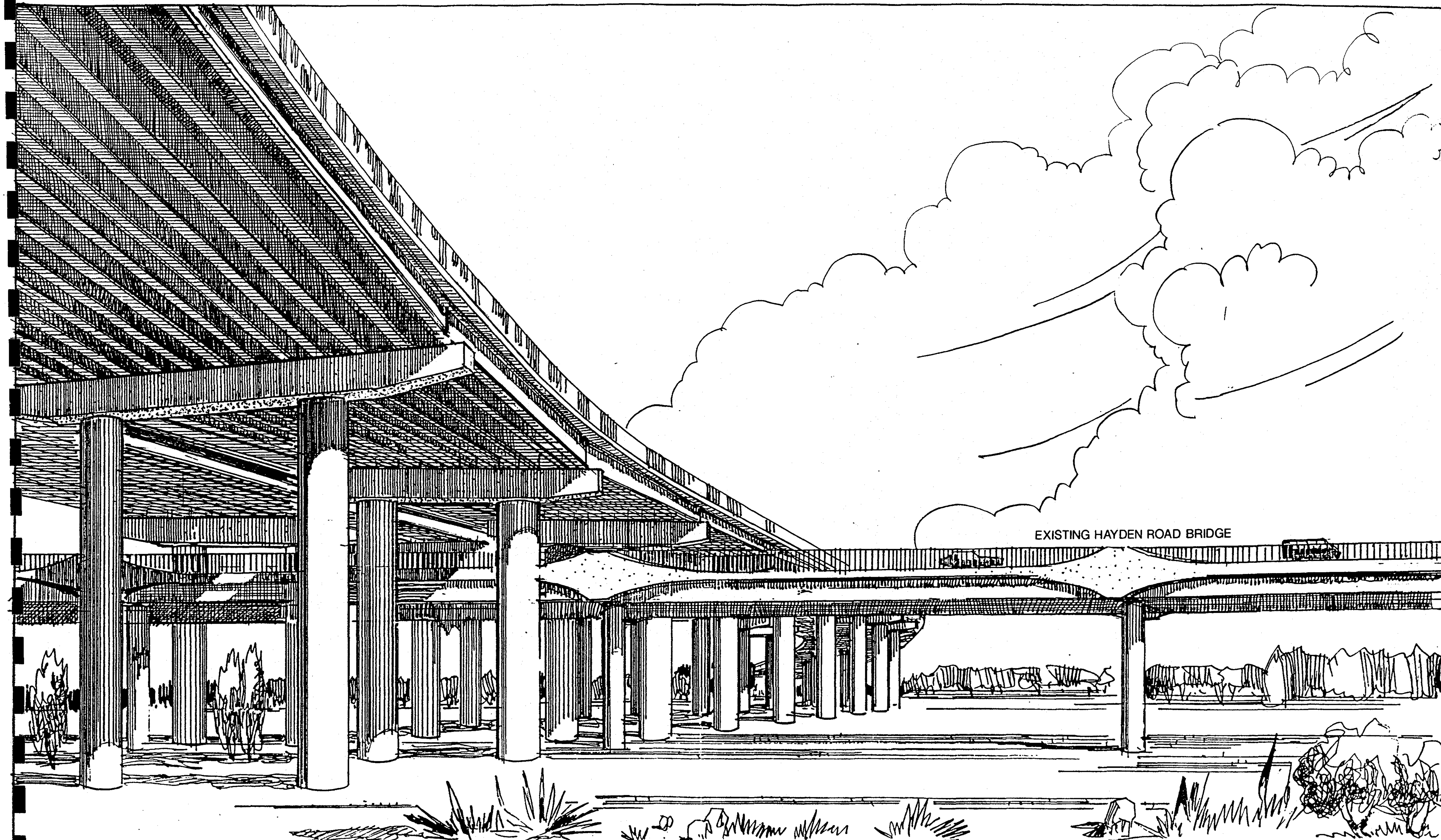
SALT RIVER BRIDGE

EAST PAPAGO FREEWAY

CURVED STEEL PLATE GIRDERS

200-FOOT SPANS

FIGURE 17



SALT RIVER BRIDGE

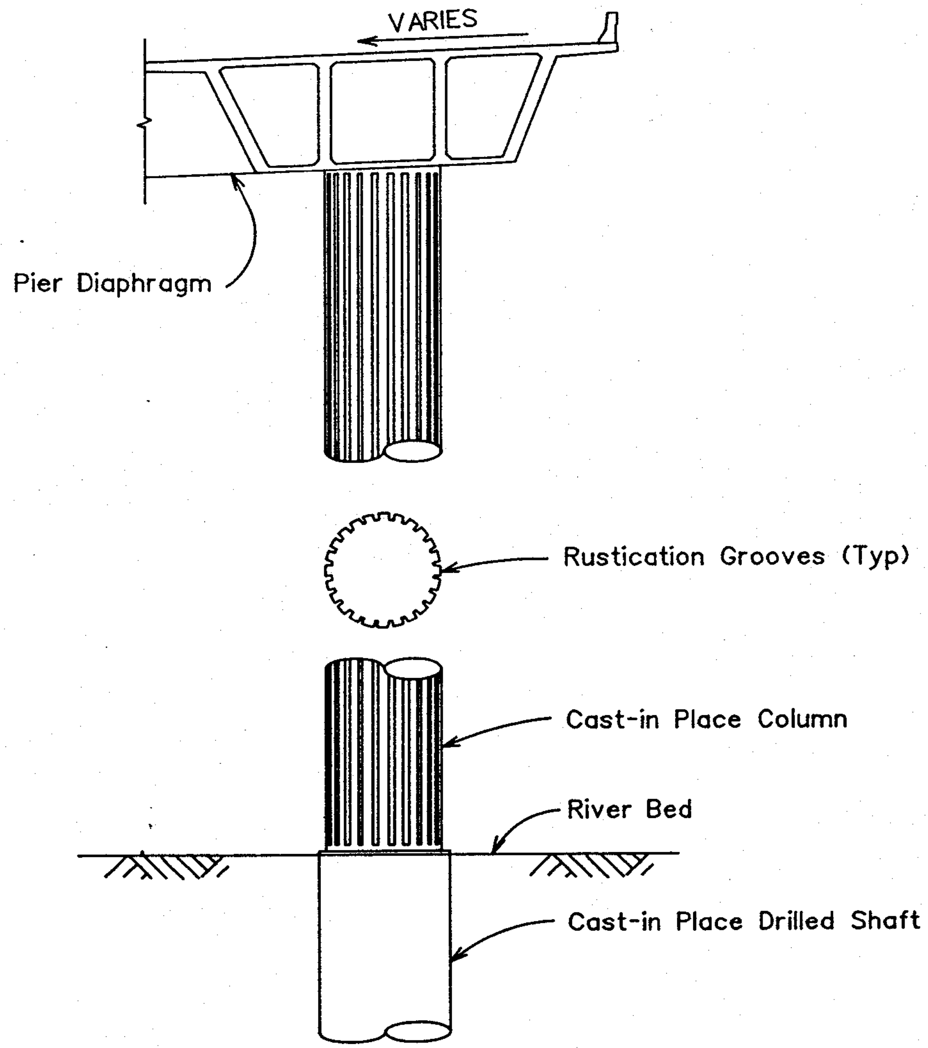
EAST PAPAGO FREEWAY

PRE-TENSIONED AASHTO TYPE VI GIRDERS

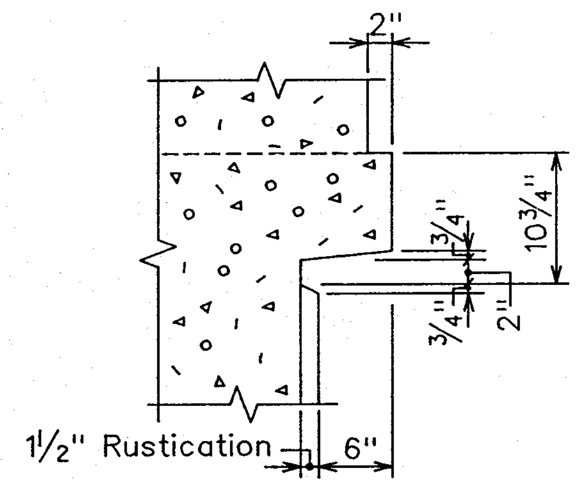
145-FOOT SPANS

FIGURE 18

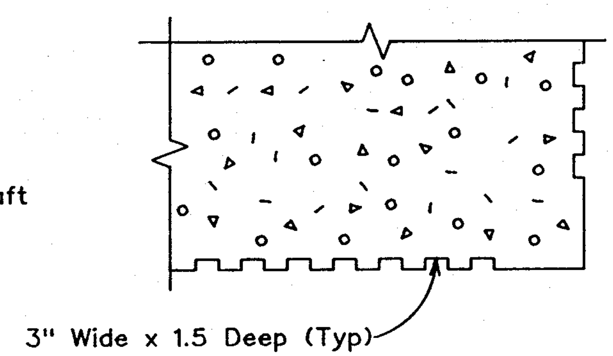
ARCHITECTURAL TREATMENT



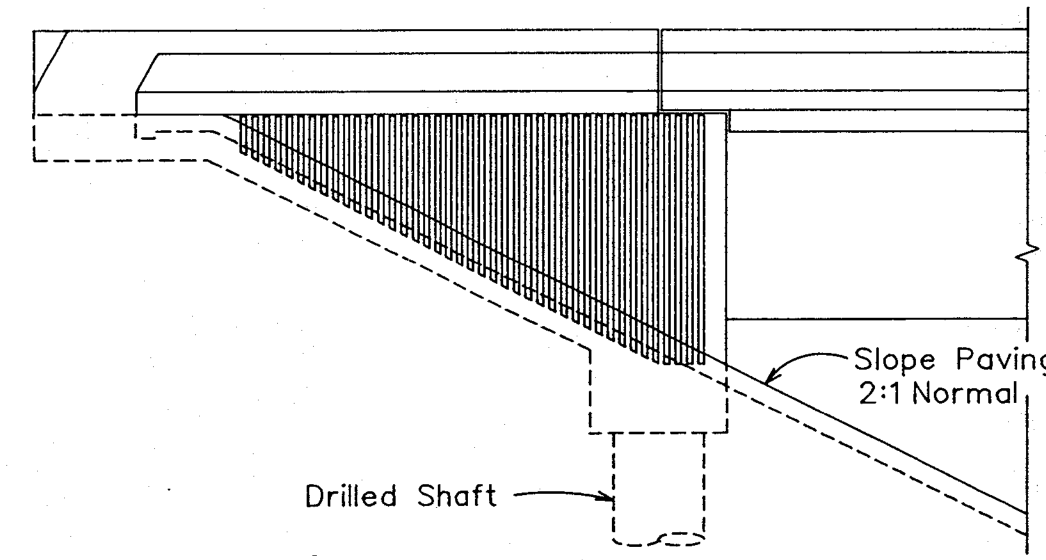
PIER COLUMN ELEVATION



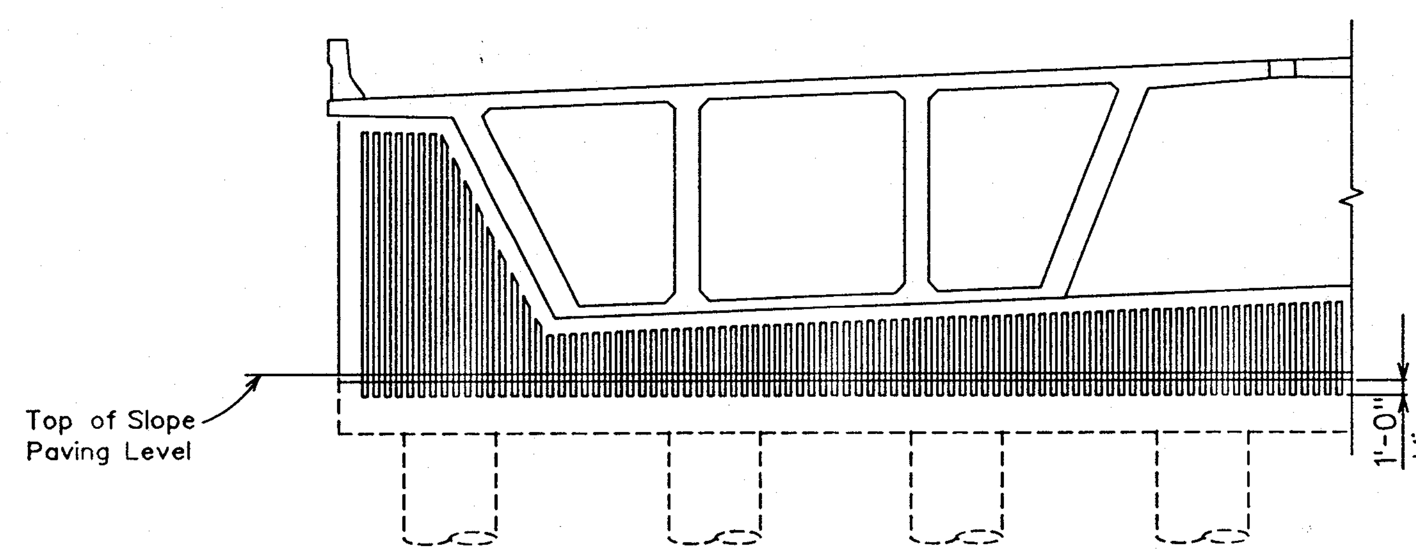
TOP OF WINGWALL DETAIL



RUSTICATION DETAIL



WINGWALL ELEVATION



ABUTMENT ELEVATION

10. COST COMPARISONS

Preliminary quantity and cost estimates were further developed for the three preferred alternates. cast-in-place twin box girders at 200 foot spans, curved steel plate girders at 200 foot spans and precast AASHTO Type VI girders at 145 foot maximum spans. The estimated quantities were derived from preliminary analyses/designs and the unit costs were obtained from contractors, material suppliers, and bid prices for recent projects.

The cost comparison matrix (Table 4) includes estimated costs itemized by primary structural components, total cost, and unit structure cost (dollars per square foot). The most ecomomical scheme is the concrete box girder bridge constructed on falsework. Next in order are curved steel plate girders and precast AASHTO Type VI girders, at cost differentials of 10 percent and 11 percent, respectively. The inclusion of construction launching girders, necessary for cast-in-place construction without erecting falsework, adds an estimated $2 million, and brings the box girder alternate up very close to the other two alternates.

Since foundation design, and associated costs, are dependent upon subsurface conditions, it should be noted that the available basis for foundation design was very limited. The drilled shaft capacity curves, developed by Sergent, Hauskins, & Beckwith, were based upon a total of six borings. All six borings were located east of Hayden Road, the deepest being about 150 feet. As currently aligned, about 40% of the bridge is situated west of Hayden Road, supported on both existing grade and on Ramp B earth fill.

Structural steel represents such a large component of the curved plate girder alternate, that the cost estimate is fairly sensitive to material unit cost: A difference of $0.10 per pound of structural steel results in a variation of $1.3 million for the scheme. Price information from numerous fabricators and bid prices for many projects were evaluated to select the rate of $0.90 per pound.

TABLE 4

East Papago - Section 6 - Salt River Bridge

PRELIMINARY QUANTITY AND COST ESTIMATES SUMMARY

Item	Unit	CIP Double PT Box Girders	Curved Steel Plate Girders	Precast AASHTO VI Girders
SUPERSTRUCTURE:				
CIP Concrete	CY	$7,081,600	$2,338,400	$2,348,600
Reinforcing	LB	$2,832,640	$935,360	$939,440
Prestressing	LB	$2,950,740	--	--
Structural Steel	LB	--	$10,653,120	--
AASHTO Type VI Girders	LF	--	--	$7,084,000
Barrier curb	LF	$253,000	$253,000	$253,000
Expansion joints	LF	$581,000	$581,000	$639,100
Bearings	Each	$25,600	$174,000	$218,400
Shear Studs	Each	--	$84,280	--
SUPERSTRUCTURE TOTAL		$13,724,580	$15,019,160	$11,482,540
SUBSTRUCTURE:				
Pier Cap Concrete	CY	--	$1,388,800	$1,637,800
Pier Cap Reinforcing	LB	--	$555,520	$655,120
Pier concrete	CY	$1,624,000	$1,353,400	$2,217,600
Pier reinforcing	LB	$733,040	$610,899	$945,500
Abutment concrete	CY	$271,800	$271,800	$271,800
Abutment reinforcing	LB	$108,720	$108,720	$108,720
Drilled shaft (10')	LF	$3,930,000	$3,180,000	$5,472,000
Drilled shaft (8')	LF	$655,000	$530,000	$570,000
Drilled shaft (4')	LF	$640,000	$640,000	$640,000
SUBSTRUCTURE TOTAL		$7,962,560	$8,639,139	$12,518,540
SUBTOTAL (STRUCTURE COST)		$21,687,140	$23,658,299	$24,001,080
MOBILIZATION	5%	$1,084,357	$1,182,915	$1,200,054
SUBTOTAL		$22,771,497	$24,841,214	$25,201,134
CONTINGENCIES	15%	$3,415,725	$3,726,182	$3,780,170
SCHEME TOTAL		**$26,187,222**	**$28,567,396**	**$28,981,304**
Constr. Launching Truss (with mobilization &contingencies)	LS	$2,415,000	--	--
UNIT PRICE PER SCHEME:				
419,742	SF			
Superstructure	$/SF	$32.70	$35.78	$27.36
Substructure	$/SF	$18.97	$20.58	$29.82
Total	$/SF	$62.39	$68.06	$69.05
Including Launching Truss:	$/SF	$68.14	--	--

11. CONCLUSIONS AND RECOMMENDATIONS

This Structure Selection Report presents the preliminary layouts developed for both the eastbound and westbound bridges, including line and grade for the six alternate superstructure types studied. It also includes substructure and foundation alternates along with geotechnical and hydraulic data, possible construction methods, architectural treatment and estimated cost comparisons. Impact of the bridge on the major utilities in the corridor was investigated.

The method of construction of the cast-in-place box girders appears to be the key to the optimum solution. The comparative cost estimates indicate that the twin post-tensioned box girder (Alternate B) will cost 10% less than the steel girder (Alternate E) and 11% less than precast AASHTO girders (Alternate F) <u>if</u> constructed on conventional falsework. The estimated cost of the three alternates is approximately the same if falsework is not permitted in the river.

The twin box girder alternate best satisfies the objective of providing an aesthetically pleasing bridge with less columns and a proven low maintenance cost record in Phoenix. This type of bridge is also estimated to be more economical than the others when constructed on falsework - a method familiar to the contractors in this area. Both precast or steel girders do have an advantage in construction over Hayden Road. The falsework necessary for CIP girders in this span will cause some disruption to traffic which would be minimal with preformed girders.

After consideration of these different bridge types, span layouts and constructability, we recommend that the twin post-tensioned concrete box girders (Alternate B) supported by twin column piers founded on single drilled shafts be selected for this East Papago Salt River Bridge. It is also our recommendation that Special Provisions be developed to control the amount of falsework permitted in the river at any time and to prescribe protective measures to be taken by the contractor. Such specifications would be based upon historical records of river flow* with the amount of vulnerable falsework varied according to seasonal risk.

* See Table 5 in the Appendix - Section 12.3.

12.1 PRELIMINARY COST ESTIMATES

STRUCTURE SELECTION STUDY
SALT RIVER BRIDGE

SUMMARY OF UNIT PRICES FOR COST ESTIMATES

Cast-in-place concrete	
Superstructure	$ 200.00 /CY
Piers	$ 200.00 /CY
Abutments	$ 150.00 /CY
Reinforcing steel	$ 0.40 /LB
Prestressing steel	$ 1.30 /LB
Structural steel	$ 0.90 /LB
AASHTO type VI girder	$ 100.00 /LF
type VI modified	$ 90.00 /LF
Drilled shafts (including reinforcing)	
15'-0" diameter	$1,200.00 /LF
12'-0" diameter	$1,000.00 /LF
10'-0" diameter	$ 750.00 /LF
8'-0" diameter	$ 500.00 /LF
6'-0" diameter	$ 300.00 /LF
4'-0" diameter	$ 200.00 /LF

East Papago - Section 6 - Salt River Bridge

PRELIMINARY QUANTITIES AND COST ESTIMATE (08AUG89)

Alternate B : CIP double PT box girder (200' span)
(10 - 8' dia piers and drilled shafts)
(40 - 10' dia piers and drilled shafts)

Item	Quantity	Unit	Unit Price	Cost
SUPERSTRUCTURE:				
Concrete	35,408	CY	$200.00	$7,081,600
Reinforcing	7,081,600	LB	$0.40	$2,832,640
Prestressing	2,269,800	LB	$1.30	$2,950,740
Barrier curb	10,120	LF	$25.00	$253,000
Expansion joints	830	LF	$700.00	$581,000
Bearings	128	Each	$200.00	$25,600
SUPERSTRUCTURE TOTAL				$13,724,580
SUBSTRUCTURE:				
Pier concrete	8,120	CY	$200.00	$1,624,000
Pier reinforcing	1,832,600	LB	$0.40	$733,040
Abutment concrete	1,812	CY	$150.00	$271,800
Abutment reinforcing	271,800	LB	$0.40	$108,720
Drilled shaft (10')	5,240	LF	$750.00	$3,930,000
Drilled shaft (8')	1310.00	LF	$500.00	$655,000
Drilled shaft (4')	3,200	LF	$200.00	$640,000
SUBSTRUCTURE TOTAL				$7,962,560
SUBTOTAL (TOTAL STRUCTURAL COST)				$21,687,140
MOBILIZATION (5%)				$1,084,357
SUBTOTAL				$22,771,497
CONTINGENCIES (15%)				$3,415,725
SCHEME TOTAL				$26,187,222
Constr. Launching Truss (+ mobilization & contingencies)	1	LS	$2,000,000	$2,415,000

UNIT PRICE PER SCHEME: (419,742 SF)

	Excluding Truss	Including Truss
Superstructure	$39.48	$45.24
Substructure	$22.91	$22.91
Total	$62.39	$68.14

East Papago - Section 6 - Salt River Bridge

PRELIMINARY QUANTITIES AND COST ESTIMATE (08AUG89)

Alternate E : Curved steel plate girder (200' span)
(10 - 8' dia piers and drilled shafts)
(40 - 10' dia piers and drilled shafts)

Item	Quantity	Unit	Unit Price	Cost
SUPERSTRUCTURE:				
Deck concrete	11,692	CY	$200.00	$2,338,400
Deck reinforcing	2,338,400	LB	$0.40	$935,360
Structural steel	11,836,800	LB	$0.90	$10,653,120
Barrier curb	10,120	LF	$25.00	$253,000
Expansion joints	830	LF	$700.00	$581,000
Bearings	174	Each	$1,000.00	$174,000
Shear Studs	60,200	Each	$1.40	$84,280
SUPERSTRUCTURE TOTAL				$15,019,160
SUBSTRUCTURE:				
Pier cap concrete	6,944	CY	$200.00	$1,388,800
Pier cap reinforcing	1,388,800	LB	$0.40	$555,520
Pier concrete	6,767	CY	$200.00	$1,353,400
Pier reinforcing	1,527,247	LB	$0.40	$610,899
Abutment concrete	1,812	CY	$150.00	$271,800
Abutment reinforcing	271,800	LB	$0.40	$108,720
Drilled shaft (10')	4,240	LB	$750.00	$3,180,000
Drilled shaft (8')	1,060	LB	$500.00	$530,000
Drilled shaft (4')	3,200	LB	$200.00	$640,000
SUBSTRUCTURE TOTAL				$8,639,139
SUBTOTAL (TOTAL STRUCTURAL COST)				$23,658,299
MOBILIZATION (5%)				$1,182,915
SUBTOTAL				$24,841,214
CONTINGENCIES (15%)				$3,726,182
SCHEME TOTAL				$28,567,396

UNIT PRICE PER SCHEME: (419,742 SF)

Superstructure	$43.21
Substructure	$24.85
Total	$68.06

East Papago - Section 6 - Salt River Bridge

PRELIMINARY QUANTITIES AND COST ESTIMATE (08AUG89)

Alternate F : Precast AASHTO Type VI girders (145' span)
(10 - 8' dia piers and drilled shafts)
(64 - 10' dia piers and drilled shafts)

Item	Quantity	Unit	Unit Price	Cost
SUPERSTRUCTURE:				
Deck concrete	9,743	CY	$200.00	$1,948,600
Deck reinforcing	1,948,600	LB	$0.40	$779,440
AASHTO type VI girder	70,840	LF	$100.00	$7,084,000
Diaphragm concrete	2,000	CY	$200.00	$400,000
Diaphragm reinforcing	400,000	LB	$0.40	$160,000
Barrier curb	10,120	LF	$25.00	$253,000
Expansion joints	913	LF	$700.00	$639,100
Bearings	1,092	Each	$200.00	$218,400
SUPERSTRUCTURE TOTAL				$11,482,540
SUBSTRUCTURE:				
Pier cap concrete	8,189	CY	$200.00	$1,637,800
Pier cap reinforcing	1,637,800	LB	$0.40	$655,120
Pier concrete	11,088	CY	$200.00	$2,217,600
Pier reinforcing	2,363,750	LB	$0.40	$945,500
Abutment concrete	1,812	CY	$150.00	$271,800
Abutment reinforcing	271,800	LB	$0.40	$108,720
Drilled shaft (10')	7,296	LF	$750.00	$5,472,000
Drilled shaft (8')	1,140	LF	$500.00	$570,000
Drilled shaft (4')	3,200	LF	$200.00	$640,000
SUBSTRUCTURE TOTAL				$12,518,540
SUBTOTAL (TOTAL STRUCTURAL COST)				$24,001,080
MOBILIZATION (5%)				$1,200,054
SUBTOTAL				$25,201,134
CONTINGENCIES (15%)				$3,780,170
SCHEME TOTAL				$28,981,304

UNIT PRICE PER SCHEME: (419,742 SF)

Superstructure	$33.03
Substructure	$36.01
Total	$69.05

12.2 GEOTECHNICAL REPORT

SERGENT, HAUSKINS & BECKWITH
Geotechnical Engineers, Inc.

3232 West Virginia Avenue
Phoenix, Arizona 85009
(602) 272-6848

TRANSMITTAL

DATE May 24, 1989

TO T.Y. LIN INTERNATIONAL
EMERSON COURT, SUITE 175
1817 NORTH SEVENTH STREET, PHOENIX, AZ 85006

ATTENTION JOSE SANCHEZ

PROJECT E. PAPAGO-HOHOKAM-SKY HARBOR FREEWAYS

JOB/PROPOSAL NO. E87-56, LETTER NO. 399

WE ARE SENDING YOU:

- [x] Attached
- [ ] Under separate cover the following:
- [ ] Boring Logs
- [ ] Calculations
- [x] Design Charts
- [ ] Progress Reports
- [ ] Laboratory Results
- [ ] Plans
- [ ] Specifications
- [ ] ______________

RECEIVED
MAY 24 1989
TYLI PHOENIX

DELIVERY BY:

- [x] Hand Delivery
- [ ] First Class Mail
- [ ] Registered Mail
- [ ] Express Mail
- [ ] Courier Service
- [ ] Other
- [ ] Return Receipt Requested

TRANSMITTED FOR:

- [ ] Review & Comment
- [ ] Approval
- [ ] Your Files/Information
- [x] As Requested

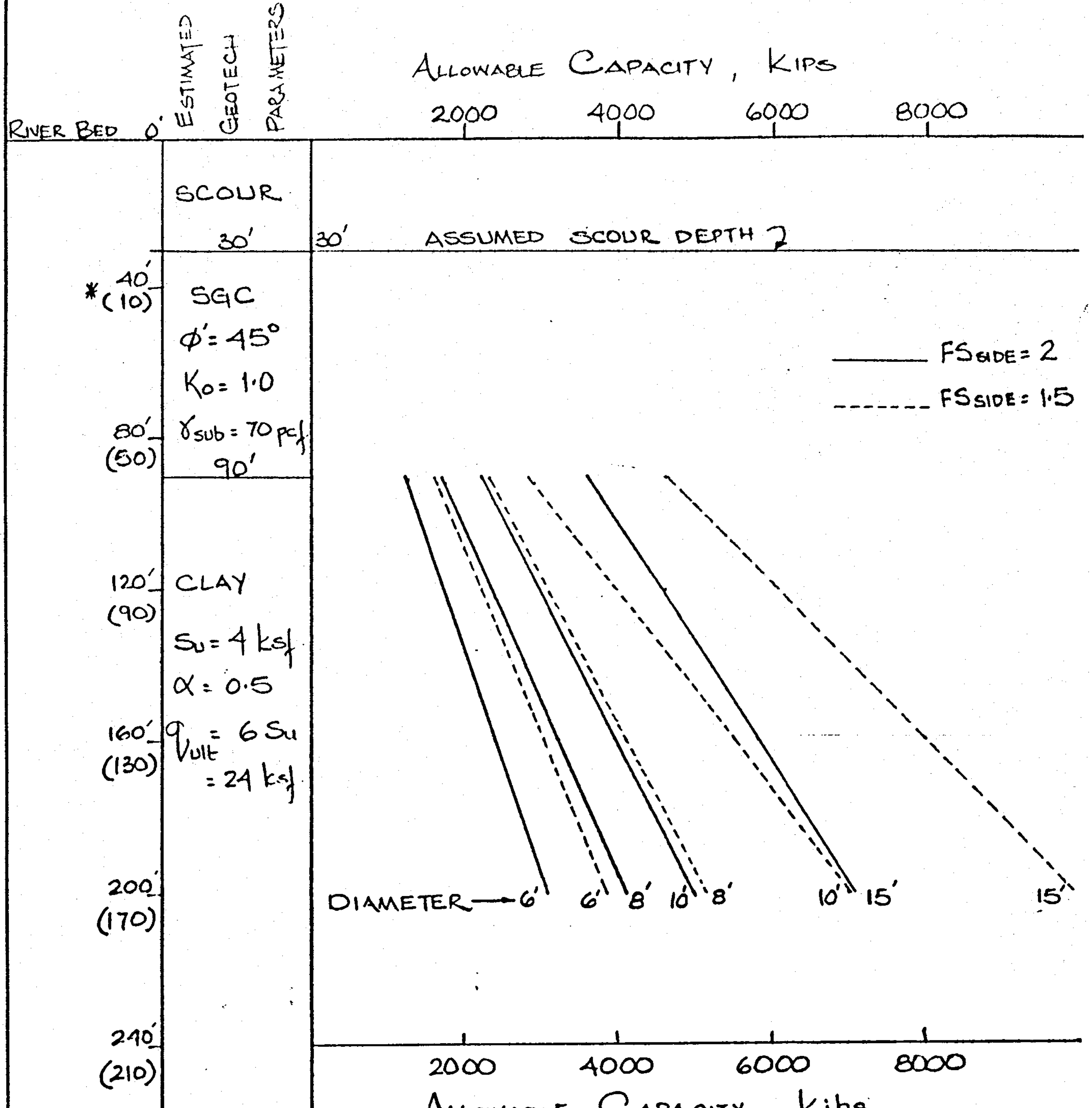
DESCRIPTION Preliminary estimates of drilled shaft capacities for Hayden Road Bridge, Design Section 6.

REMARKS Please note that these estimated capacities are for preliminary design & cost estimates only and should not be used for final design.

COPY TO DMJM
ATTN: DEL MILLER, P.E.

SIGNED Anwar Wissa

HAYDEN ROAD BRIDGE DRILLED SHAFT CAPACITY ESTIMATES



* Depth below scour in ( )

THESE ESTIMATED CAPACITIES SHALL BE REVISED AFTER FINAL GEOTECHNICAL INVESTIGATIONS ARE COMPLETED

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX • TUCSON • ALBUQUERQUE • SANTA FE • SALT LAKE CITY • EL PASO • RENO/SPARKS

Project E. PAPAGO-HOHOKAM-SKY HARBOR
Job No: E87-56
Computed by: AH Ckd. by:
Date 5.24.87 Page ___ of ___

IX SUMMARY

No. of shafts per bent	REQUIRED DEPTH BELOW SCOUR					
	DIAMETER = 6.0'		DIAMETER = 8.0'		DIAMETER = 15.0'	
	FS_{TIP} = 2.0 FS_{SIDE} = 2.0	2.0 1.5	2.0 2.0	2.0 1.5	2.0 2.0	2.0 1.5
1	–	–	–	–	197'	130'
2	–	–	167'	118'	–	–
3	144'	103'	103'	72'	–	–
4	103'	74'	–	–	–	–

THE ABOVE DEPTHS WERE EVALUATED FROM ESTIMATED SOIL PARAMETERS. THESE DEPTHS ARE PRELIMINARY AND SHOULD NOT BE USED FOR FINAL DESIGN.

THIS INFORMATION IS PROVIDED FOR PRELIMINARY DESIGN & COST ESTIMATES ONLY.

(HAYDEN ROAD BRIDGE)

MEMORANDUM

Date: July 20, 1989

To: Turan Ceran, P.E.

Copy: Mitchell Smith, P.E.
T.Y. Lin International

From: Anwar Hirany, Ph.D. AH 7/20

Re: East Papago - Hohokam -
Sky Harbor Freeways
ADOT Project No. 202L MA H 0855 01D
Arizona Department of Transportation
Maricopa County, Arizona
SHB Job No. E87-56
Letter No. 422

Subject: Drilled Shaft Capacities for Salt River Bridge at Hayden Road (Design Section 6)

We have completed three additional test borings for the Salt River Bridge at Hayden Road in Design Section 6 of the referenced project. Pressuremeter tests were performed in the borings at selected intervals and relatively undisturbed soil samples were obtained with a Dennison soil sampler. These borings were drilled to a depth of approximately 150 feet below grade.

Intermittent layers of sand and clay were encountered at depths below approximately 90 feet in the test borings. The limit pressures obtained from the pressuremeter tests indicated that this stratified soil layer generally is very stiff to hard. However, laboratory tests performed on clay samples obtained with the Dennison sampler indicate that the clay consistency varied from medium to stiff. According to observations made by field personnel, intermittent layers of hard and comparatively soft layers were encountered during drilling between sampling and pressuremeter testing intervals.

Prior to drilling these additional borings, we had recommended that an ultimate tip resistance (end-bearing) of 24 kips per square foot (ksf) be used for capacity determination to evaluate preliminary cost estimates for the foundations. However, based on the results of these additional field and laboratory tests, we now recommend that an ultimate tip resistance of 12 ksf be used for preliminary cost estimates of drilled piers founded more than 90 feet below existing grade. To evaluate side resistance below 90 feet below existing grade, an undrained shear strength of 4 ksf and an adhesion factor of 0.5 is recommended. Because of the coarse nature of the sand, gravel and cobbles (SGC) encountered above 90 feet, an average overbreak of one foot is recommended for evaluating side resistance in this layer.

Our analysis show that drilled shaft capacities evaluated with the criteria recommended above give comparable values to those shown in Figure 1, which was transmitted to DMJM and T.Y. Lin International on May 24, 1989 (Letter No. 399). This is because overbreak in the SGC was not accounted for in our preliminary analysis for developing the capacity curves shown in the figure. The soil parameters used for developing the capacity curves shown in Figure 1 were estimated from field penetration resistance tests performed in test borings drilled for the referenced bridge prior to May 24, 1989. These estimated soil parameters are also shown in the figure.

It should be noted that the capacity curves were extrapolated to a depth of 200 feet on the assumption that the clay layer extends beyond that depth. The validity of this assumption should be verified during the supplementary geotechnical investigations to be conducted by the Section Designer.

12.3 HYDROTECHNICAL REPORT

sla

SIMONS, LI & ASSOCIATES, INC.

4600 S. MILL AVENUE, SUITE 280
TEMPE, ARIZONA 85282
TELEPHONE (602) 491-1393

Dennis L. Richards, P.E.
Assistant Vice President

June 5, 1989

Mr. Mitchell D. Smith, P.E.
T.Y. Lin International
1817 N. Seventh Street
Suite 175
Phoenix, Arizona 85006

Re: Local Scour Estimates for Various Pier Configurations, East Papago Freeway Crossing of the Salt River.

Dear Mitch:

This letter summarizes our hydraulic analyses and local scour estimates for various pier configurations for the East Papago Freeway crossing of the Salt River. Based on our May 18, 1989 meeting, we have analyzed different pier configuarations and evaluated drilled cylindrical columns versus piers placed on piles with a 34-foot diameter pile cap.

Three different pier configurations were examined: 1) piers with three 6-foot diameter columns per structure; 2) piers with two 8-foot diameter columns per structure; and 3) piers with one 15-foot diameter column per structure. The piers in each case were assumed to be on 200-foot centers and skewed sufficiently to expose each column to the flow. Due to the large skew of the bridge crossing, it is impractical from a structural standpoint to align the piers with the flow.

Computed 100-year water surface elevations for each case are given in Table 1. The results show that the maximum water surface elevations occur with the use of the 6-foot diameter piers, but the maximum difference between these elevations and those computed for the other pier configurations is only 0.1 feet. This small difference is due to the similar projected width of the different pier configurations when skew is considered. The projected width is 18 feet for the three 6-foot diameter piers, 16 feet for the two 8-foot diameter piers, and 15 feet for the single 15-foot pier.

The results of the local scour estimates for each pier configuration are given in Table 2. Included in the table are estimates of scour with a 6-foot debris blockage added to each pier. The estimates given are the most conservative of several pier scour equations reported in the literature and used for this analysis. None of the equations explicitly account for armoring during the scour process, which could limit the depth of scour. Although we routinely consider the armoring process in our sediment routing studies, we are unaware of an adequate means of considering the armor potential for local scour around bridge piers.

Fort Collins, CO • Tempe, AZ • Tucson, AZ • Newport Beach, CA

The above scour estimates assume drilled cylindrical piers to a depth below the scour hole. Drilled cylindrical piers have been used with success on alluvial streams in Arizona where local scour potential is very large. In order to obtain the total scour depth for each pier configuration, 6 feet should be added to the local scour depth. This additional depth is to account for general scour and bed forms.

We also investigated scour potential for piers placed on piles with a 34-foot diameter pile cap. Due to the large local scour depths computed, it would appear impractical to place the pile caps below the potential scour depth. There is no method presently available to estimate scour around a pile cap located in or above the scour zone. It is possible that the pile cap may act as a scour arrestor, blocking the horseshoe vortex and reducing the depth of scour. However, if the pile cap were exposed to the flow sufficiently, it is possible that local scour would be increased due to an increase in the effective width of the pier. Using a width of 34 feet in the pier scour equations results in a scour depth of 50 feet. While it is unlikely this depth would be obtained, the potential for local scour is significant. A good estimate of the depth of scour for this situation could only be determined by a physical model study. If completely exposing the pile caps during a flood is unacceptable from a structural standpoint, we recommend the use of drilled cylindrical piers to a depth below the scour potential.

If you have any questions regarding the scour values or need additional information, please feel free to contact me.

Sincerely,

SIMONS, LI & ASSOCIATES, INC.

Dennis L. Richards

Dennis L. Richards, P.E.
Assistant Vice President

DLR:klw
Enclosures
PAZ-DMJM-03/PH/L13
cc: Mike Ports

Table 1. Water-Surface Elevations for 100-Year Peak Discharge, East Papago Freeway Crossing.

River* Distance (ft)	Water-Surface Elevation 6-ft Piers (ft)	8-ft Piers (ft)	15-ft Piers (ft)
0	1,172.44	1,172.44	1,172.44
206	1,172.94	1,172.91	1,172.90
295	1,173.70	1,173.67	1,173.66
615	1,174.33	1,174.31	1,174.29
1,015	1,174.84	1,174.78	1,174.75
1,415	1,175.35	1,175.28	1,175.25
1,815	1,175.50	1,175.43	1,175.40
2,235	1,175.77	1,175.70	1,175.67
2,635	1,175.78	1,175.71	1,175.68
3,035	1,175.82	1,175.76	1,175.73
3,445	1,176.04	1,175.99	1,175.96
3,855	1,176.97	1,176.93	1,176.90
4,255	1,178.51	1,178.48	1,178.46
4,755	1,180.90	1,180.88	1,180.87

* River Distance 295 is the upstream face of the Hayden Road Bridge.

Table 2. Summary of Pier Scour Estimates, East Papago Freeway Crossing.

Pier Diameter (ft)	Local Scour (ft)	Local Scour With 6-ft Debris Blockage (ft)
6	16	25
8	19	27
15	29	36

sla

SIMONS, LI & ASSOCIATES, INC.

FAX PHONE: 1-602-491-1396

DATE: 8/4/89

PLEASE DELIVER THIS TO MR. MITCHELL SMITH

RE: EAST PAPAGO CROSSING OF THE SALT RIVER

MR SMITH:

HERE IS THE DESIGN DISCHARGES AND CALCULATED WATER SURFACE ELEVATIONS FOR A TWO HUNDRED Ft. SPAN CONFIGURATION. WE WILL FORMALLY TRANSMIT THIS INFORMATION IN LETTER FORM ASAP. INCLUDED IS A MAP OF THE CROSS-SECTION LOCATIONS. CROSS-SECTION 227.10 IS THE DOWNSTREAM FACE OF THE EXISTING HAYDEN RD. BRIDGE. IF THERE ARE ANY FURTHER QUESTIONS, PLEASE FEEL FREE TO CALL.

RECEIVED
AUG 4 1989
TYLI PHOENIX

THERE ARE 3 PAGES TO THIS TRANSMISSION (INCLUDING THIS COVER PAGE).
PLEASE CALL 602-491-1393 IF YOU HAVE NOT RECEIVED ALL PAGES.

SIGNED Jeff Minch

4600 South Mill Avenue, Suite 280, Tempe, Arizona 85282

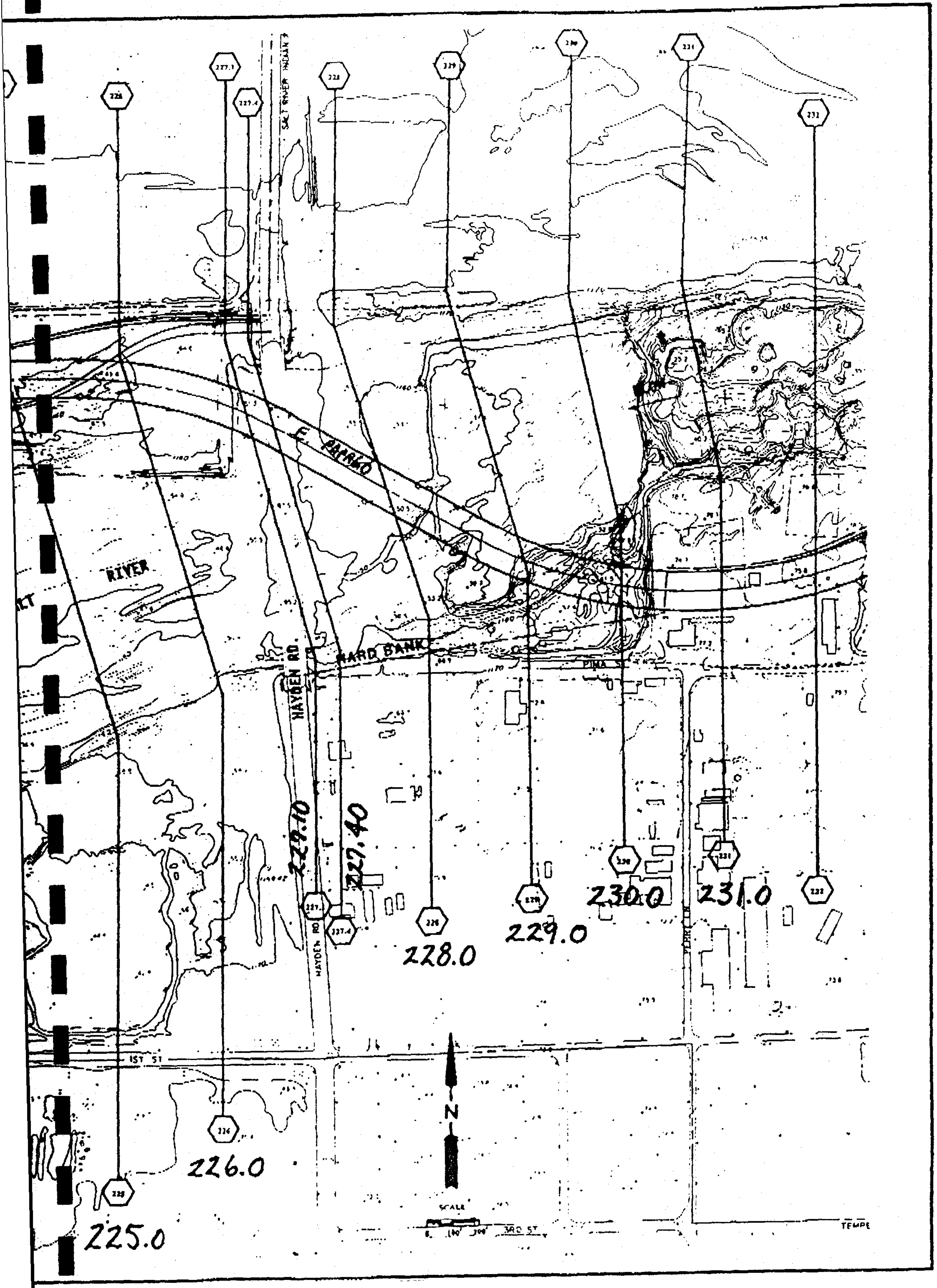
East Papago Crossing of the Salt River

(200 ft. Spans with 2-10 ft. dia. columns per structure)

Design Discharge: Q_{100} = 215,000 cfs (100-Year Event)

Q_{SPF} = 289,000 cfs (Standard Project Flood)

CROSS-SECTION NUMBER	WATER SURFACE ELEVATIONS (ft) 100-YEAR	SPF
225.0	1170.65	1173.38
226.0	1171.52	1174.42
227.1	1172.97	1176.29
227.4	1173.72	1176.99
227.0	1174.18	1177.39
228.0	1174.46	1177.76
229.0	1175.00	1178.48
230.0	1175.42	1179.34
231.0	1175.57	1179.47



SLA Simons, Li & Associates, Inc.
Newport Beach, CA., Fort Collins, CO.
Tucson, AZ., Phoenix, AZ.

HEC-2 Cross-Section Locations

Project No.
Date:
Design:
Drawn:
Check:
Revisions:

Plate 1
3 of 4

sla

SIMONS, LI & ASSOCIATES, INC.

4600 S. MILL AVENUE, SUITE 280
TEMPE, ARIZONA 85282
TELEPHONE (602) 491-1393

Dennis L. Richards, P.E.
Assistant Vice President

August 7, 1989

RECEIVED
AUG 8 1989
TYLI PHOENIX

Mr. Mitchell D. Smith, P.E.
T.Y. LIN International
Emerson Court, Suite 270
1817 North Seventh Street
Phoenix, Arizona 85006

Re: Hydraulic Data for the East Papago Crossing of the Salt River

Dear Mitch:

This letter summarizes SLA's hydraulic analysis of a 200 ft span configuration for the East Papago crossing of the Salt River. It was assumed that each structure would be supported by two 10-ft. diameter columns that would be aligned perpendicular to the roadway.

Discharges used in the hydraulic analysis are based on values presented in the May 1982 Central Arizona Water Control Study (U.S. Army Corps of Engineers). The 100-year frequency analysis used a design discharge of 215,000 cfs. The standard project flood analysis used a design discharge of 289,000 cfs. Water surface elevations and average velocities are presented in Table 1.

Included with this letter is a copy of the Salt River Project's data base of average daily releases. If you have any questions or need additional information, please feel free to contact me.

Sincerely,

SIMONS, LI & ASSOCIATES, INC.

Dennis L. Richards

Dennis L. Richards, P.E.
Vice President

DLR:gc
Enclosures
TYLIN2.WP

Data Furnished by Simons, Li & Associates, Inc. from Salt River Project Files.

TABLE 5

SPILLS FROM GRANITE REEF AND
RELEASES FROM DRAINS ABOVE HAYDEN ROAD

OBS	DATE	SPILL
1	08/01/64	2,764
2	08/27/64	524
3	04/20/65	3,590
4	04/21/65	2,320
5	04/22/65	3,360
6	04/23/65	804
7	12/22/65	1,900
8	12/23/65	6,900
9	12/24/65	4,300
10	12/25/65	2,300
11	12/26/65	2,100
12	12/27/65	990
13	12/30/65	6,100
14	12/31/65	64,000
15	01/01/66	53,000
16	01/02/66	17,000
17	01/03/66	11,000
18	01/04/66	12,000
19	01/05/66	12,000
20	01/06/66	13,000
21	01/07/66	13,000
22	01/08/66	13,000
23	01/09/66	12,000
24	01/10/66	11,000
25	01/11/66	1,000
26	02/13/66	555
27	02/14/66	524
28	02/20/66	1,082
29	02/21/66	1,594
30	02/22/66	2,280
31	02/23/66	2,308
32	02/24/66	1,835
33	02/25/66	1,385
34	02/26/66	1,379
35	02/27/66	1,450
36	02/28/66	1,469
37	03/01/66	1,332
38	03/02/66	1,231
39	03/03/66	1,241
40	09/13/66	2,158
41	07/17/67	550
42	12/19/67	2,450
43	12/20/67	2,963
44	02/14/68	1,632
45	02/15/68	3,703
46	02/16/68	3,471
47	02/17/68	3,437
48	02/18/68	3,408
49	02/19/68	1,357
50	02/25/68	1,573
51	02/26/68	2,957
52	02/27/68	2,603
53	02/28/68	2,542
54	02/29/68	2,508
55	03/01/68	1,125
56	03/09/68	555
57	03/10/68	1,062
58	03/11/68	1,074
59	03/12/68	1,821
60	03/13/68	3,319
61	03/14/68	2,629
62	03/15/68	760
63	04/14/68	644
64	04/15/68	969
65	04/16/68	1,479
66	04/17/68	1,521
67	04/18/68	1,448
68	04/19/68	1,349
69	04/20/68	1,264
70	04/21/68	1,242
71	04/22/68	842
72	10/04/72	501
73	10/07/72	5,310
74	10/19/72	9,294
75	10/20/72	4,053
76	10/21/72	1,032
77	11/23/72	592
78	11/24/72	526
79	12/12/72	1,045
80	12/13/72	1,030
81	12/28/72	1,648
82	12/29/72	5,493
83	12/30/72	5,273
84	12/31/72	850
85	01/01/73	610
86	01/02/73	719
87	01/03/73	1,355
88	01/04/73	1,336
89	01/05/73	1,616
90	01/06/73	671
91	02/21/73	1,350
92	02/22/73	2,767
93	02/23/73	4,380
94	02/24/73	2,722
95	02/25/73	2,753
96	02/26/73	2,104
97	03/03/73	1,407
98	03/04/73	1,677
99	03/05/73	1,776
100	03/06/73	1,671
101	03/07/73	1,367
102	03/12/73	1,869
103	03/13/73	5,815
104	03/14/73	9,957
105	03/15/73	8,685
106	03/16/73	7,109
107	03/17/73	9,012
108	03/18/73	12,435
109	03/19/73	13,102
110	03/20/73	13,003
111	03/21/73	11,625
112	03/22/73	8,509
113	03/23/73	9,271
114	03/24/73	12,657
115	03/25/73	12,931
116	03/26/73	10,266
117	03/27/73	8,445
118	03/28/73	8,386
119	03/29/73	11,215
120	03/30/73	15,184
121	03/31/73	18,711
122	04/01/73	22,321
123	04/02/73	20,329
124	04/03/73	14,475
125	04/04/73	11,861
126	04/05/73	6,995
127	04/06/73	6,434
128	04/07/73	6,411
129	04/08/73	6,293
130	04/09/73	5,593
131	04/10/73	4,752
132	04/11/73	4,619
133	04/12/73	5,835
134	04/13/73	11,728
135	04/14/73	12,318
136	04/15/73	13,348
137	04/16/73	13,993
138	04/17/73	14,625
139	04/18/73	14,126
140	04/19/73	14,044
141	04/20/73	13,627
142	04/21/73	9,108
143	04/22/73	9,102
144	04/23/73	8,976
145	04/24/73	7,791
146	04/26/73	1,123
147	04/27/73	870
148	04/28/73	5,019
149	04/29/73	5,617
150	04/30/73	5,977
151	05/01/73	4,878
152	05/02/73	5,534
153	05/03/73	5,313
154	05/04/73	4,781
155	05/05/73	1,821
156	05/06/73	5,871

DRAINS INCLUDE HENNESEY, EVERGREEN, AND TEMPE

III

SPILLS FROM GRANITE REEF AND
RELEASES FROM DRAINS ABOVE HAYDEN ROAD

OBS	DATE	SPILL	OBS	DATE	SPILL	OBS	DATE	SPILL
157	05/07/73	10,929	209	01/01/79	2,693	261	03/22/79	10,000
158	05/08/73	9,232	210	01/02/79	2,684	262	03/23/79	10,980
159	05/09/73	4,782	211	01/03/79	2,395	263	03/24/79	11,950
160	05/10/73	4,533	212	01/04/79	2,322	264	03/25/79	11,970
161	05/11/73	5,997	213	01/05/79	7,955	265	03/26/79	11,720
162	05/12/73	6,769	214	01/06/79	6,600	266	03/27/79	14,970
163	05/13/73	6,348	215	01/07/79	1,200	267	03/28/79	20,075
164	05/14/73	5,828	216	01/17/79	36,213	268	03/29/79	51,803
165	05/15/73	5,948	217	01/18/79	87,546	269	03/30/79	40,600
166	05/16/73	5,785	218	01/19/79	70,112	270	03/31/79	46,460
167	05/17/73	5,657	219	01/20/79	56,805	271	04/01/79	22,400
168	05/18/73	4,298	220	01/21/79	26,705	272	04/02/79	11,800
169	05/19/73	1,316	221	01/22/79	20,310	273	04/03/79	9,830
170	05/22/73	1,609	222	01/23/79	19,100	274	04/04/79	11,140
171	05/23/73	1,148	223	01/24/79	17,300	275	04/05/79	7,270
172	02/28/78	985	224	01/25/79	16,200	276	04/06/79	2,060
173	03/01/78	4,088	225	01/26/79	16,000	277	04/07/79	2,780
174	03/02/78	70,809	226	01/27/79	14,100	278	04/08/79	6,100
175	03/03/78	95,809	227	01/28/79	14,200	279	04/09/79	6,830
176	03/04/78	37,000	228	01/29/79	13,400	280	04/10/79	12,460
177	03/05/78	22,798	229	01/30/79	9,900	281	04/11/79	13,500
178	03/06/78	19,520	230	01/31/79	10,200	282	04/12/79	13,735
179	03/07/78	16,700	231	02/01/79	10,490	283	04/13/79	14,609
180	03/16/78	754	232	02/02/79	8,800	284	04/14/79	13,080
181	03/17/78	1,858	233	02/03/79	8,800	285	04/15/79	12,000
182	03/18/78	1,521	234	02/04/79	8,800	286	04/16/79	2,715
183	03/19/78	1,750	235	02/05/79	8,000	287	05/01/79	1,040
184	03/20/78	1,750	236	02/06/79	4,000	288	05/02/79	1,050
185	03/21/78	1,761	237	02/07/79	3,800	289	05/03/79	500
186	03/22/78	1,592	238	02/08/79	3,800	290	05/04/79	620
187	03/23/78	6,963	239	02/09/79	3,800	291	05/05/79	600
188	03/24/78	5,543	240	02/10/79	3,800	292	05/06/79	520
189	03/25/78	2,120	241	02/11/79	3,800	293	01/30/80	3,750
190	03/26/78	1,900	242	02/12/79	2,500	294	01/31/80	6,025
191	03/27/78	1,895	243	02/13/79	2,200	295	02/01/80	9,300
192	03/28/78	500	244	02/14/79	1,800	296	02/02/80	8,485
193	03/31/78	1,746	245	02/15/79	1,600	297	02/03/80	8,275
194	12/17/78	500	246	02/16/79	504	298	02/04/80	7,000
195	12/18/78	30,800	247	02/17/79	504	299	02/05/80	4,545
196	12/19/78	110,000	248	02/18/79	504	300	02/06/80	4,345
197	12/20/78	88,300	249	02/19/79	504	301	02/07/80	1,308
198	12/21/78	59,400	250	03/11/79	765	302	02/08/80	4,245
199	12/22/78	35,000	251	03/12/79	3,334	303	02/09/80	1,980
200	12/23/78	31,000	252	03/13/79	3,800	304	02/14/80	9,350
201	12/24/78	9,400	253	03/14/79	4,144	305	02/15/80	89,024
202	12/25/78	9,400	254	03/15/79	9,597	306	02/16/80	139,132
203	12/26/78	7,000	255	03/16/79	9,945	307	02/17/80	67,719
204	12/27/78	6,500	256	03/17/79	9,752	308	02/18/80	72,270
205	12/28/78	5,500	257	03/18/79	9,752	309	02/19/80	53,783
206	12/29/78	4,400	258	03/19/79	7,352	310	02/20/80	55,458
207	12/30/78	2,400	259	03/20/79	6,680	311	02/21/80	82,484
208	12/31/78	1,500	260	03/21/79	6,654	312	02/22/80	89,640

DRAINS INCLUDE HENNESEY, EVERGREEN, AND TEMPE

SPILLS FROM GRANITE REEF AND
RELEASES FROM DRAINS ABOVE HAYDEN ROAD

OBS	DATE	SPILL
313	02/23/80	54,730
314	02/24/80	52,693
315	02/25/80	49,003
316	02/26/80	18,389
317	02/27/80	15,101
318	02/28/80	14,347
319	02/29/80	11,231
320	03/01/80	11,199
321	03/02/80	11,000
322	03/03/80	11,000
323	03/04/80	7,824
324	03/05/80	4,941
325	03/06/80	700
326	03/27/80	900
327	03/28/80	1,700
328	03/29/80	1,460
329	03/30/80	1,660
330	03/31/80	1,770
331	04/01/80	1,670
332	04/02/80	1,685
333	04/03/80	2,060
334	04/04/80	2,375
335	04/05/80	2,340
336	04/06/80	2,520
337	04/07/80	2,500
338	04/08/80	2,200
339	04/09/80	2,100
340	04/10/80	1,980
341	04/11/80	1,970
342	04/12/80	1,890
343	04/13/80	1,945
344	04/14/80	1,885
345	04/15/80	840
346	04/16/80	650
347	04/17/80	575
348	04/18/80	575
349	04/19/80	915
350	04/20/80	950
351	04/21/80	960
352	04/22/80	950
353	04/23/80	995
354	04/24/80	600
355	04/30/80	695
356	05/01/80	2,435
357	05/02/80	2,555
358	05/03/80	2,420
359	05/04/80	2,400
360	05/05/80	1,500
361	03/13/82	1,625
362	03/14/82	9,017
363	03/15/82	8,825
364	03/16/82	8,819
365	03/17/82	5,860
366	03/18/82	1,947
367	03/28/82	784
368	03/29/82	3,159
369	12/01/82	581
370	12/11/82	2,153
371	12/12/82	2,368
372	12/13/82	2,247
373	12/14/82	2,068
374	12/15/82	2,091
375	12/16/82	2,093
376	12/17/82	1,894
377	12/24/82	2,007
378	12/25/82	2,203
379	12/26/82	2,184
380	12/27/82	2,595
381	12/28/82	6,385
382	12/29/82	6,283
383	12/30/82	6,215
384	12/31/82	4,992
385	01/01/83	1,889
386	01/02/83	1,814
387	01/03/83	1,948
388	01/04/83	1,479
389	01/05/83	1,190
390	01/06/83	612
391	01/30/83	716
392	02/04/83	2,307
393	02/05/83	4,871
394	02/06/83	4,672
395	02/07/83	5,017
396	02/08/83	11,294
397	02/09/83	30,014
398	02/10/83	30,441
399	02/11/83	25,852
400	02/12/83	24,760
401	02/13/83	15,979
402	02/14/83	11,692
403	02/15/83	8,167
404	02/16/83	7,994
405	02/17/83	7,343
406	02/18/83	5,752
407	02/19/83	2,877
408	02/20/83	2,880
409	02/21/83	2,880
410	02/22/83	2,532
411	02/23/83	2,355
412	02/24/83	1,907
413	02/25/83	1,668
414	02/26/83	1,700
415	02/27/83	1,821
416	02/28/83	2,041
417	03/01/83	2,056
418	03/02/83	2,818
419	03/03/83	5,212
420	03/04/83	12,215
421	03/05/83	13,195
422	03/06/83	13,075
423	03/07/83	11,413
424	03/08/83	11,125
425	03/09/83	4,385
426	03/10/83	3,838
427	03/11/83	2,814
428	03/12/83	2,569
429	03/13/83	2,548
430	03/14/83	2,546
431	03/15/83	2,138
432	03/16/83	2,028
433	03/17/83	1,135
434	03/18/83	894
435	03/19/83	587
436	03/20/83	3,517
437	03/21/83	9,894
438	03/22/83	17,952
439	03/23/83	16,392
440	03/24/83	16,775
441	03/25/83	19,706
442	03/26/83	20,372
443	03/27/83	14,866
444	03/28/83	6,345
445	03/29/83	5,562
446	03/30/83	5,359
447	03/31/83	8,556
448	04/01/83	12,304
449	04/02/83	11,892
450	04/03/83	9,120
451	04/04/83	7,491
452	04/05/83	6,776
453	04/06/83	5,803
454	04/07/83	3,936
455	04/08/83	3,054
456	04/09/83	1,591
457	04/10/83	1,358
458	04/11/83	1,002
459	04/12/83	581
460	04/13/83	840
461	04/14/83	885
462	04/15/83	1,172
463	04/16/83	1,240
464	04/17/83	1,242
465	04/18/83	941
466	04/19/83	1,200
467	04/20/83	3,225
468	04/21/83	4,323

DRAINS INCLUDE HENNESEY, EVERGREEN, AND TEMPE

SPILLS FROM GRANITE REEF AND
RELEASES FROM DRAINS ABOVE HAYDEN ROAD

OBS	DATE	SPILL
469	04/22/83	7,703
470	04/23/83	8,055
471	04/24/83	7,992
472	04/25/83	6,093
473	04/26/83	5,287
474	04/27/83	3,517
475	04/28/83	3,775
476	04/29/83	2,954
477	04/30/83	2,216
478	05/01/83	2,073
479	05/02/83	2,349
480	05/03/83	1,744
481	05/12/83	1,244
482	05/13/83	1,586
483	05/14/83	1,100
484	05/15/83	770
485	05/16/83	515
486	08/16/83	706
487	10/01/83	1,957
488	10/02/83	39,408
489	10/03/83	39,976
490	10/04/83	36,469
491	10/05/83	27,411
492	10/06/83	15,479
493	10/07/83	8,841
494	10/08/83	7,477
495	10/09/83	6,857
496	10/10/83	4,390
497	10/11/83	3,573
498	10/12/83	3,350
499	10/13/83	3,790
500	10/14/83	3,822
501	10/15/83	3,797
502	10/16/83	3,656
503	10/17/83	3,962
504	10/18/83	4,597
505	10/19/83	4,747
506	10/20/83	4,797
507	10/21/83	4,866
508	10/22/83	2,367
509	12/25/83	4,600
510	12/26/83	11,200
511	12/27/83	11,067
512	12/28/83	10,317
513	12/29/83	8,271
514	12/30/83	5,634
515	12/31/83	2,106
516	01/01/84	2,088
517	01/02/84	2,169
518	01/03/84	2,013
519	01/04/84	1,655
520	01/05/84	1,643
521	01/06/84	1,663
522	01/07/84	1,724
523	01/08/84	1,360
524	01/09/84	1,251
525	01/10/84	1,168
526	01/11/84	1,186
527	01/12/84	1,245
528	01/13/84	1,224
529	01/14/84	1,168
530	01/15/84	1,153
531	01/16/84	1,003
532	10/03/84	549
533	12/20/84	807
534	12/21/84	791
535	12/22/84	1,622
536	12/23/84	3,516
537	12/24/84	3,539
538	12/25/84	5,801
539	12/26/84	4,086
540	12/27/84	7,200
541	12/28/84	26,010
542	12/29/84	24,405
543	12/30/84	23,353
544	12/31/84	22,264
545	01/01/85	15,131
546	01/02/85	13,505
547	01/03/85	3,982
548	01/04/85	2,105
549	01/05/85	1,218
550	01/06/85	1,056
551	01/07/85	920
552	01/08/85	1,182
553	01/09/85	1,609
554	01/10/85	1,633
555	01/11/85	1,652
556	01/12/85	1,634
557	01/13/85	1,615
558	01/14/85	1,609
559	01/15/85	1,599
560	01/16/85	1,538
561	01/17/85	1,435
562	01/18/85	1,406
563	01/19/85	1,427
564	01/20/85	1,484
565	01/21/85	1,474
566	01/22/85	1,389
567	01/23/85	1,346
568	01/24/85	1,345
569	01/25/85	1,336
570	01/27/85	733
571	01/28/85	6,920
572	01/29/85	8,353
573	01/30/85	6,588
574	01/31/85	6,372
575	02/01/85	5,823
576	02/02/85	4,615
577	02/03/85	4,619
578	02/04/85	4,694
579	02/05/85	5,140
580	02/06/85	3,977
581	02/07/85	2,404
582	02/08/85	2,419
583	02/09/85	2,394
584	02/10/85	2,372
585	02/11/85	2,411
586	02/12/85	2,169
587	02/13/85	1,644
588	02/14/85	2,217
589	02/15/85	2,019
590	02/24/85	2,308
591	02/25/85	2,985
592	02/26/85	3,800
593	02/27/85	3,765
594	02/28/85	3,695
595	03/01/85	4,741
596	03/02/85	13,232
597	03/03/85	10,816
598	03/04/85	6,839
599	03/05/85	3,817
600	03/06/85	2,875
601	03/07/85	2,720
602	03/08/85	2,068
603	03/09/85	1,317
604	03/10/85	1,305
605	03/11/85	1,321
606	03/12/85	2,226
607	03/13/85	9,378
608	03/14/85	9,919
609	03/15/85	13,119
610	03/16/85	14,195
611	03/17/85	14,689
612	03/18/85	16,731
613	03/19/85	14,229
614	03/20/85	2,494
615	03/21/85	4,441
616	03/22/85	2,331
617	03/23/85	2,288
618	03/24/85	2,298
619	03/25/85	2,226
620	03/26/85	1,359
621	03/27/85	1,300
622	03/28/85	1,301
623	03/29/85	1,274
624	03/30/85	1,450

DRAINS INCLUDE HENNESEY, EVERGREEN, AND TEMPE

SPILLS FROM GRANITE REEF AND RELEASES FROM DRAINS ABOVE HAYDEN ROAD

OBS	DATE	SPILL
625	03/31/85	1,450
626	04/01/85	1,602
627	04/02/85	2,975
628	04/03/85	2,428
629	04/04/85	1,100
630	04/05/85	1,100
631	04/06/85	975
632	04/07/85	1,050
633	04/08/85	1,693
634	04/09/85	2,544
635	04/10/85	2,292
636	04/11/85	1,500
637	04/12/85	1,500
638	04/13/85	1,500
639	04/14/85	1,333
640	04/15/85	1,156
641	04/16/85	750
642	04/23/85	660
643	04/24/85	2,268
644	04/25/85	1,195
645	04/28/85	707
646	04/29/85	1,989
647	04/30/85	2,454
648	05/01/85	2,608
649	05/02/85	2,975
650	05/03/85	2,390
651	05/04/85	2,178
652	05/05/85	1,986
653	05/06/85	1,229
654	05/07/85	617
655	12/09/85	672
656	12/10/85	1,996
657	12/11/85	2,103
658	12/12/85	2,046
659	12/13/85	2,120
660	12/14/85	2,124
661	12/15/85	1,929
662	12/16/85	2,035
663	12/17/85	2,034
664	12/18/85	1,865
665	12/19/85	1,490
666	12/20/85	1,454
667	12/21/85	1,432
668	12/22/85	1,423
669	12/23/85	1,438
670	12/24/85	1,477
671	12/25/85	1,482
672	12/26/85	1,459
673	12/27/85	1,401
674	12/28/85	1,360
675	12/29/85	1,277
676	12/30/85	1,312
677	12/31/85	1,067
678	04/05/86	930
679	04/06/86	789

OBS	DATE	SPILL
365	12/11/85	2103
366	12/12/85	2046
367	12/13/85	2120
368	12/14/85	2124
369	12/15/85	1929
370	12/16/85	2035
371	12/17/85	2034
372	12/18/85	1865
373	12/19/85	1490
374	12/20/85	1454
375	12/21/85	1432
376	12/22/85	1423
377	12/23/85	1438
378	12/24/85	1477
379	12/25/85	1482
380	12/26/85	1459
381	12/27/85	1401
382	12/28/85	1360
383	12/29/85	1277
384	12/30/85	1312
385	12/31/85	1067
386	04/05/86	930
387	04/06/86	789
388	03/17/87	1433
389	03/18/87	1358
390	03/19/87	1438
391	03/20/87	1972
392	03/21/87	2723
393	03/22/87	2898
394	03/23/87	2108
395	03/24/87	1258
396	03/25/87	620

03/26/87	292
3/27/87	82
3/28/87	42
3/29/87	20
3/30/87	102
3/31/87	157
4/1/87	44
4/2/87	15
4/3/87	15
4/4/87	22
4/5/87	37
4/6/87	20
4/7/87	12
4/8/87	20
4/9/87	20
4/10/87	8
4/11 → 15	5

Thru: June 24

DRAINS INCLUDE HENNESEY, EVERGREEN, AND TEMPE

12.4 SELECTED CALCULATIONS

T.Y. LIN INTERNATIONAL STRUCTURAL ENGINEERING 1817 N. 7th St., Suite 270, Phoenix, AZ 85006	PROJECT: EAST PAPAGO SECTION 6
	ITEM: SALT RIVER BRIDGE
	DESIGN: [illegible]
	DATE: [illegible]

SHEET: ____ OF ____
REVISION:

ADOT MULTICELL BOX

SPAN	DEPTH	ASPECT RATIO
240'	12'	20
200'	10'	20
180'	8.5'	21.2
160'	7.5'	21.3

TRIAL CROSS SECTION (9-CELL) 80' ROADWAY

CANTILEVER
$L \cong 6'-0"$
$T_{min} = 9"$
$T_{max} = 1'-3"$

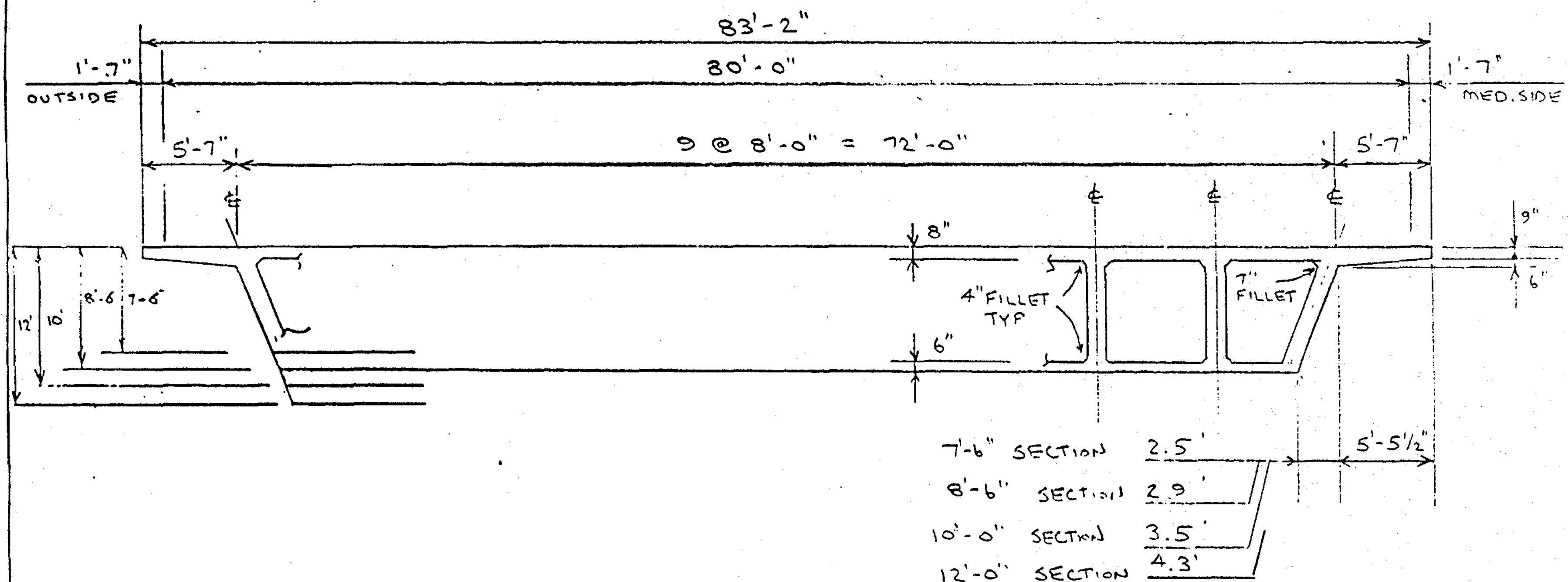
BARRIERS
1'-7" MEDIAN SIDE
1'-7" OUTSIDE
} USE 0.4 KLF × 2

DECK
(BASED ON 7'-9" ± C-C WEB SPACING)
8" TOTAL THICKNESS

SOFFIT
6"
HAUNCH THICKNESS = 12" MAX
LENGTH = 20'-0" FROM ℄ SUPPORT

WEBS
EXTERIOR 2.5 : 1 NOMINAL SIDE SLOPE
14" THICKNESS
FLARE THICKNESS = 24" MAX
LENGTH = 20'-0" FROM ℄ SUPPORT

FILLETS
4" TYPICAL INTERIOR WEBS
7" TOP OF EXTERIOR WEBS ONLY
(ie. WEIGHTED AVG. SIZE = 4.33" TOP , 4" BOT)



TYPICAL MIDSPAN SECTION

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
1817 N. 7th St. Suite 270, Phoenix, AZ 85006

PROJECT: EAST PAPAGO SECTION 6
ITEM: SALT RIVER BRIDGE
DESIGN:
DATE:

SHEET:
OF
REVISION:

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
1817 N. 7th St., Suite 270, Phoenix, AZ 85006

PROJECT:	EAST PAPAGO
ITEM:	SALT RIVER BRIDGE
DESIGN:	([illegible]) SUPERSTRUCTURE
DATE:	[illegible]

SHEET:
OF
REVISION:

LIVE LOAD LANES = 80/14 = 5.71

MATERIALS (CONCRETE)

USE f_c' = 5000 SUPERSTRUCTURE E = 580393 KSF
= 3500 SUBSTRUCTURE E = 485592 KSF

SECTION PROPERTY NOTES.

FOR STRENGTH CONSIDERATIONS, USE A, I NEGLECTING 1/2" OF DECK

FOR SHEAR AREA, USE (WEB TH.)(NO. OF WEBS)(DEPTH − 1/2")

FOR DEAD LOAD, USE FULL CROSS SECTIONAL AREA

FOR FORMWORK LEFT IN PLACE USE (CELL WIDTH)(10 PSF)

INTERMEDIATE DIAPHRAGMS USE t = 9"

PIER DIAPHRAGMS: USE 10'-0" FOR ALL SCHEMES
TO MATCH MAX. COLUMN SIZE.

DIAPHRAGM SELF WEIGHT = (CELL AREA)(TH.)(150 PCF)

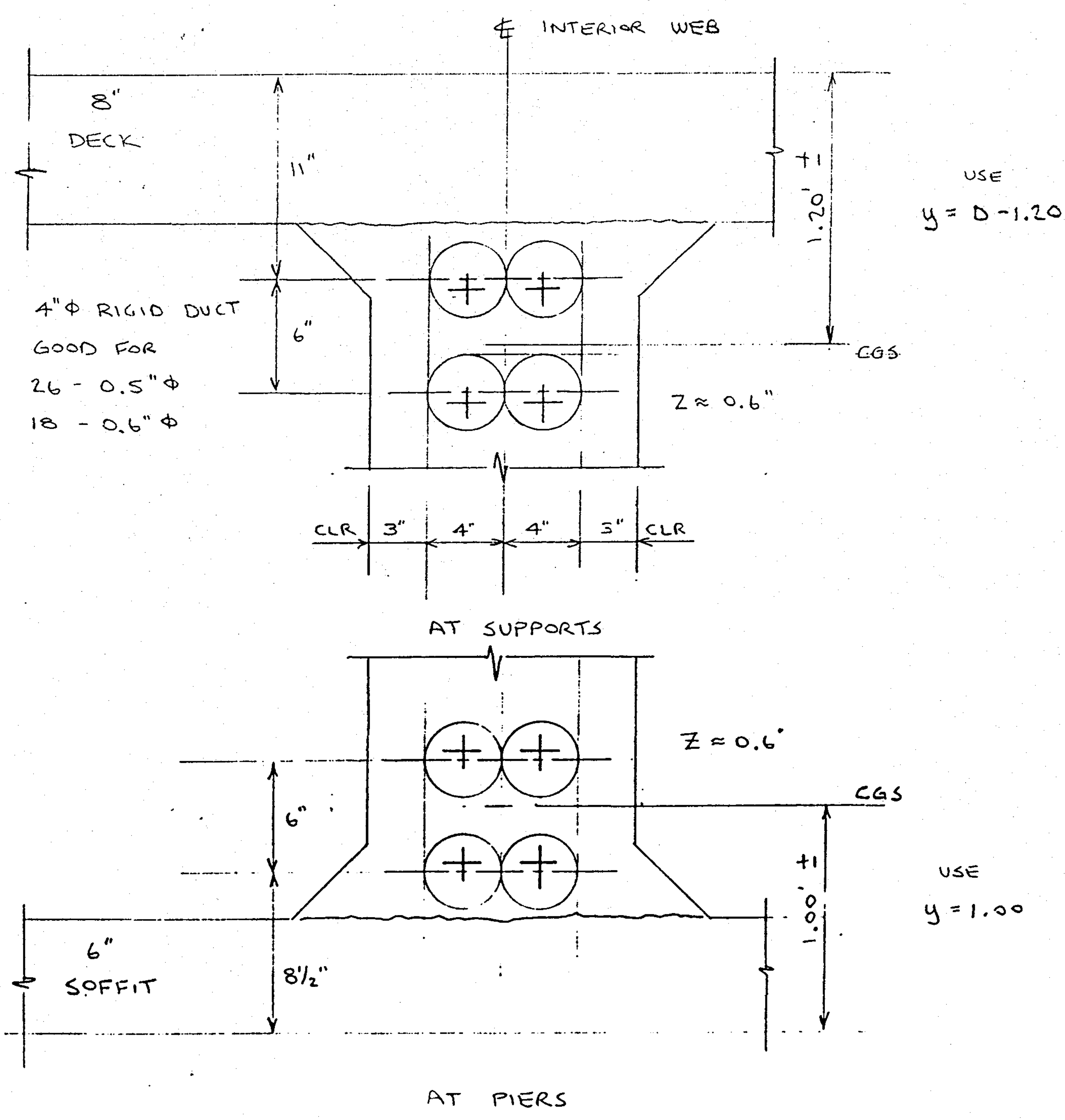
FOR LINK ELEMENT USE A = 68' × (TH.) = A_V
$I = 1/12\,(68')(TH.)^3$

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
1817 N. 7th St., Suite 270, Phoenix, AZ 85006

PROJECT: EAST PAPAGO SECTION 6
ITEM: SALT RIVER BRIDGE
DESIGN: [illegible]
DATE: [illegible] 83 DR[illegible]

SHEET:
OF
REVISION:

PRESTRESS GEOMETRY (ASSUMED MAX ACHIEVABLE ECCENTRICITIES)



INTERIOR WEB SHOWN : EXTERIOR WEB SIMILAR

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
1817 N. 7th St., Suite 270, Phoenix, AZ 85006

PROJECT:	EAST PAPAGO SECTION 6
ITEM:	SALT RIVER BRIDGE
DESIGN:	(INITIAL) SUPERSTRUCTURE
DATE:	28 APR 89 DRC

SHEET:
OF
REVISION:

EQUIVALENT WOBBLE COEFFICIENT
(TO ACCOUNT FOR HORIZONTAL CURVE)

K = WOBBLE FRICTION COEFFICIENT = 0.0002 FT^{-1}
μ = CURVATURE FRICTION COEFFICIENT = 0.25 RAD^{-1}

$K_{EQUIVALENT} = K + \mu$ (HORIZONTAL CURVATURE)

MAINLINE ALIGNMENT CURVES ARE 03° 30' 00" CURVES

$$\frac{3.5}{100} \cdot \frac{\pi}{180} = 0.00061 \text{ RAD/FT}$$

$$K_{EQUIVALENT} = 0.0002 + 0.25\,(0.00061)$$
$$= 0.00035$$

USE $K = 0.0003$ FOR CURVED AND TANGENT SECTIONS.

Project: 00534 E.Papago Sec.6
Item: Salt River Bridge
Design: Section Properties
Date: 05MAY89 Dan Shiosaka

SECTION PROPERTIES CALCULATIONS

Salt River Bridge

Midspan Section For 10'-0" Depth

Input	Units	
Width	feet	82.88
Depth	feet	10.00
Length Of Cantilever	feet	6.00
Edge (min.) Thickness Of Cantilever	inches	9.00
Support (max.) Thickness Of Cantilever	inches	15.00
Exterior Web Side Slope	X(V):1(H)	2.50
Deck Thickness	inches	8.50
Soffit Thickness	inches	6.00
Number Of Interior Webs	#	7
Exterior Web Thickness	inches	14.00
Interior Web Thickness	inches	14.00
Upper Fillet Length	feet	0.36
Upper Fillet Thickness	inches	4.33
Lower Fillet Length	feet	0.33
Lower Fillet Thickness	inches	4.00

b	d	A	y	Ay	Ay2	Io
82.88	10.00	828.75	5.00	4143.75	20718.75	6906.25
-12.00	0.50	-3.00	8.92	-26.75	-238.52	-0.04
-12.00	8.75	-105.00	4.38	-459.38	-2009.77	-669.92
-7.00	8.75	-30.63	2.92	-89.32	-260.53	-130.26
-7.03	8.79	-30.92	6.36	-196.67	-1251.03	-132.76
-53.60	8.79	-471.19	4.90	-2306.88	-11294.08	-3035.00
5.77	0.36	1.04	9.17	9.55	87.61	0.01
4.67	0.33	0.78	0.61	0.48	0.29	0.00
		189.84		1074.79	5752.73	2938.27

8691.01
-6085.09

yb = 5.66 Feet
yt = 4.34 Feet

I = 2605.92 Ft4

Sb = 460.27 Ft3
St = 600.67 Ft3

Project: 00534 E.Papago Sec.6
Item: Salt River Bridge
Design: Section Properties
Date: 05MAY89 Dan Shiosaka

SECTION PROPERTIES CALCULATIONS

Salt River Bridge

Midspan Section For 10'-0" Depth

Input * (Neglect top 1/2" of deck)	Units	
Width	feet	82.88
* Depth	feet	9.96
Length Of Cantilever	feet	6.00
* Edge (min.) Thickness Of Cantilever	inches	8.50
* Support (max.) Thickness Of Cantilever	inches	14.50
Exterior Web Side Slope	X(V):1(H)	2.50
* Deck Thickness	inches	8.00
Soffit Thickness	inches	6.00
Number Of Interior Webs	#	7.00
Exterior Web Thickness	inches	14.00
Interior Web Thickness	inches	14.00
Upper Fillet Length	feet	0.36
Upper Fillet Thickness	inches	4.33
Lower Fillet Length	feet	0.33
Lower Fillet Thickness	inches	4.00

b	d	A	y	Ay	Ay2	Io
82.88	9.96	825.30	4.98	4109.29	20460.84	6820.28
-12.00	0.50	-3.00	8.92	-26.75	-238.52	-0.04
-12.00	8.75	-105.00	4.38	-459.38	-2009.77	-669.92
-7.00	8.75	-30.63	2.92	-89.32	-260.53	-130.26
-7.03	8.79	-30.92	6.36	-196.67	-1251.03	-132.76
-53.60	8.79	-471.19	4.90	-2306.88	-11294.08	-3035.00
5.77	0.36	1.04	9.17	9.55	87.61	0.01
4.67	0.33	0.78	0.61	0.48	0.29	0.00
		186.38		1040.33	5494.83	2852.31

8347.13
-5806.77

yb = 5.58 Feet
yt = 4.38 Feet

I = 2540.36 Ft4

Sb = 455.12 Ft3
St = 580.43 Ft3

BRIDGE DESIGN : STRESSES ON SUPERSTRUCTURE

Sht.___of____
T.Y. Lin Int.l
Run by:_____ Date:________
Checked by:_____ Date:________

PROJECT :East Papago Section 6 "STRCKINP.DAT"

BRIDGE :Salt River Viaduct Preliminary Analysis 08MAY89 Dan Shiosaka

3 x 200 FT SPANS

COMBINATION	ALLOWABLE STRESSES IN KIPS PER SQUARE FEET COMPRESSION	TENSION	%
P0+D1	277.2	.0	100
P+D	288.0	.0	100
P+D+LMAX	288.0	30.5	100
P+D+LMIN	288.0	30.5	100
P+D+TMAX	403.2	42.0	140
P+D+TMIN	403.2	42.0	140
P+D+LMAX+TMAX	360.0	30.2	125
P+D+LMIN+TMIN	360.0	30.2	125
P+D+SC	288.0	30.5	100

```
    1          2          3
---------------------------
         |          |
         |          |
        -|-        -|-
         1          2
```

SEGMENT	CURRENT		RECOMMENDED CHANGES		
	PRESTRESS LEVEL	JACKING SEQUENCE	PRESTRESS LEVEL	JACKING SEQUENCE	OTHER
1					

BRIDGE DESIGN : STRESSES ON SUPERSTRUCTURE

Sht.___of___
T.Y. Lin Int.l
Run by:_____ Date:________
Checked by:_____ Date:________

PROJECT :East Papago Section 6 "STRCKINP.DAT"

BRIDGE :Salt River Viaduct Preliminary Analysis 08MAY89 Dan Shiosaka

3x 200 FT SPANS

S U M M A R Y   O F   S T R E S S E S

			PRESTRESS				DEAD LOADS									
POINT	JOINT	HO	HO/A	M(PO)	S(PO)	S(P)	M(D1)	S(D1)	M(D2)	S(D2)	M(D3)	S(D3)	M(SC)	S(SC)	M(D)	S(D)
Span1	3	-27080.	-145.	-100580.	28.	24.	80366.	-138.	7804.	-13.	3344.	-6.	-966.	2.	88170.	-152.
					-366.	-319.		177.		17.		7.		-2.		194.
Span1	4	-27080.	-145.	-95700.	20.	17.	75336.	-130.	7406.	-13.	3174.	-5.	-1610.	3.	82742.	-143.
					-356.	-309.		166.		16.		7.		-4.		182.
Pr1LT	7	-27080.	-113.	109507.	-286.	-249.	-135739.	215.	-13188.	21.	-5652.	9.	-3220.	5.	-148926.	235.
					33.	29.		-181.		-18.		-8.		-4.		-199.
Pr1RT	7	-24360.	-101.	97842.	-256.	-223.	-105876.	167.	-10289.	16.	-4409.	7.	2134.	-3.	-116164.	184.
					29.	25.		-141.		-14.		-6.		3.		-155.
Span2	10	-24360.	-131.	-73762.	-4.	-3.	37264.	-64.	3711.	-6.	1591.	-3.	2134.	-4.	40976.	-71.
					-293.	-255.		82.		8.		3.		5.		90.
Pr2LT	13	-24360.	-101.	107905.	-272.	-237.	-105875.	167.	-10289.	16.	-4409.	7.	2134.	-3.	-116164.	184.
					42.	37.		-141.		-14.		-6.		3.		-155.
Pr2RT	13	-27051.	-113.	117080.	-299.	-260.	-135739.	215.	-13188.	21.	-5652.	9.	-3220.	5.	-148926.	235.
					45.	39.		-181.		-18.		-8.		-4.		-199.
Span3	16	-27051.	-145.	-98600.	25.	22.	75336.	-130.	7406.	-13.	3174.	-5.	-1610.	3.	82742.	-143.
					-362.	-315.		166.		16.		7.		-4.		182.
Span3	17	-27051.	-145.	-106579.	38.	33.	80366.	-138.	7804.	-13.	3344.	-6.	-966.	2.	88170.	-152.
					-379.	-330.		177.		17.		7.		-2.		194.

BRIDGE DESIGN : STRESSES ON SUPERSTRUCTURE

Sht.___of___
T.Y. Lin Int.l
Run by:_____ Date:________
Checked by:_____ Date:________

PROJECT :East Papago Section 6 "STRCKINP.DAT"

BRIDGE :Salt River Viaduct Preliminary Analysis 08MAY89 Dan Shiosaka

3 x 200 FT SPANS

POINT	ENVELOPES FOR LIVE LOADS LMAX M	LMAX S	LMIN M	LMIN S	TEMPERATURE TMAX M	TMAX S	TMIN M	TMIN S	STRESS COMBINATIONS P0+DI	P+D	P+D+ LMAX	P+D+ LMIN	P+D+ TMAX	P+D+ TMIN	P+D+ LMAX+ TMAX	P+D+ LMIN+ TMIN	P+D+ SC
Span1	16102.	-28.	-1286.	2.	1907.	-3.	-3508.	6.	-116.	-128.	-155.	-125.	-131.	-122.	-159.	-119.	-126.
		35.		-3.		4.		-8.	-182.	-125.	-90.	-128.	-121.	-133.	-85.	-135.	-127.
Span1	16025.	-28.	-2143.	4.	3178.	-5.	-5847.	10.	-116.	-125.	-153.	-122.	-131.	-115.	-158.	-112.	-123.
		35.		-5.		7.		-13.	-183.	-128.	-92.	-132.	-121.	-141.	-85.	-145.	-131.
Pr1LT	-23898.	38.	0.	0.	-11694.	18.	6356.	-10.	-62.	-13.	25.	-13.	5.	-23.	43.	-23.	-8.
		-32.		0.		-16.		8.	-155.	-170.	-201.	-170.	-185.	-161.	-217.	-161.	-174.
Pr1RT	-23856.	38.	0.	0.	-4212.	7.	7751.	-12.	-82.	-39.	-1.	-39.	-32.	-51.	5.	-51.	-42.
		-32.		0.		-6.		10.	-118.	-130.	-161.	-130.	-135.	-119.	-167.	-119.	-127.
Span2	12362.	-21.	-4042.	7.	7750.	-13.	-4212.	7.	-71.	-74.	-95.	-67.	-87.	-66.	-108.	-60.	-77.
		27.		-9.		17.		-9.	-207.	-165.	-138.	-174.	-148.	-174.	-120.	-183.	-160.
Pr2LT	-23856.	38.	0.	0.	-4211.	7.	7749.	-12.	-98.	-53.	-15.	-53.	-46.	-65.	-9.	-65.	-56.
		-32.		0.		-6.		10.	-105.	-118.	-150.	-118.	-123.	-108.	-155.	-108.	-115.
Pr2RT	-23897.	38.	0.	0.	-11693.	18.	6355.	-10.	-75.	-25.	13.	-25.	-6.	-35.	32.	-35.	-20.
		-32.		0.		-16.		8.	-144.	-160.	-192.	-160.	-175.	-151.	-207.	-151.	-164.
Span3	16025.	-28.	-2143.	4.	3178.	-5.	-5847.	10.	-111.	-121.	-149.	-117.	-126.	-111.	-154.	-107.	-118.
		35.		-5.		7.		-13.	-189.	-133.	-98.	-138.	-126.	-146.	-91.	-151.	-137.
Span3	16102.	-28.	-1286.	2.	1907.	-3.	-3508.	6.	-106.	-118.	-146.	-116.	-122.	-112.	-149.	-110.	-117.
		35.		-3.		4.		-8.	-195.	-136.	-101.	-139.	-132.	-144.	-97.	-147.	-138.

P0 = INITIAL PRESTRESS (H0 = AXIAL FORCE) , P = FINAL PRESTRESS
D1 = BOX GIRDER DEAD LOAD + PERMANENT FORMWORK , D2 = BARRIERS + FUTURE WEARING SUFACE , D3 = TEMPORARY CONSTRUCTION LOAD
DI = INITIAL DEAD LOAD = D1 + D3 , D = FINAL DEAD LOAD = D1 + D2
L = LIVE LOAD ENVELOPES
T = TEMPERATURE LOAD ENVELOPES
SC = SHRINKAGE AND CREEP
LONG TERM LOSSES = 13.00%
ALL STRESSES ARE IN KIPS PER SQUARE FEET

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

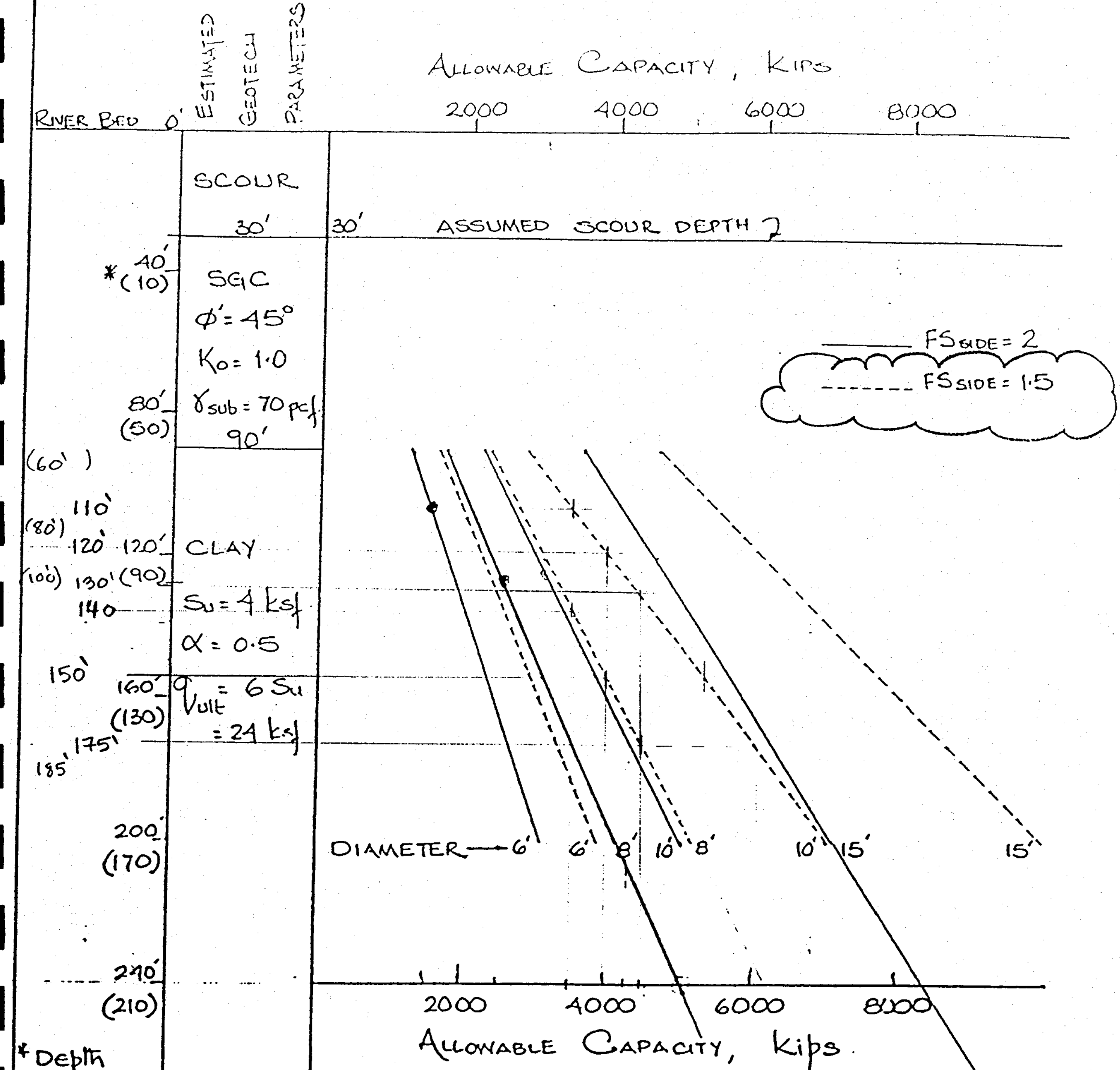
PROJECT: SALT RIVER BRIDGE
ITEM: DRILLED SHAFT REACTIONS
DESIGN: PRELIMINARY SUMMARY
DATE: [illegible]

SHEET: ____ OF ____
REVISION:

REACTION (KIPS) \ SPAN (FT)		160	190	200	240
DL 1		5100	6000	6900	9500
DL 2		450	510	570	760
DL 3		200	220	250	330
LL_{max} (NO Impact)		760	840	1020	1560
P_ϕ		150	210	60	NEGL.
T		190	230	80	30
Total Reaction		6,850	8,010	8,880	12,180
Load/COLUMN		3,425	4,005	4,440	6,090
LENGTH OF SHAFTS (FT)	8' ϕ	140 *	150 *	175	240 *
	10' ϕ	110	120	130	175

* SPANS OVER R AND B ARE 190 FT MAX, THEREFORE USE REACTIONS FOR 200 FT SPANS TO DETERMINE LENGTH OF SHAFTS.

SALT RIVER BRIDGE DRILLED SHAFT CAPACITY ESTIMATES



THESE ESTIMATED CAPACITIES SHALL BE REVISED AFTER FINAL GEOTECHNICAL INVESTIGATIONS ARE COMPLETED

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX • TUCSON • ALBUQUERQUE • SANTA FE • SALT LAKE CITY • EL PASO • RENO/SPARKS

Project: E. PAPAGO-HOHOKAM-SKY HARBOR
Job No: E87-56
Computed by: AH Ckd. by:
Date: 5.24.87 Page ___ of ___

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

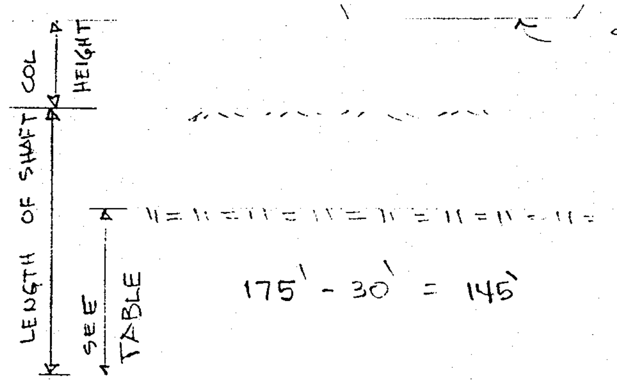
PROJECT: SALT RIVER BRIDGE
ITEM: Cost estimate : Substructure
DESIGN: R. Tinsman
DATE:

SHEET:
OF
REVISION:

160 FT SPANS : TOTAL NO. OF COLUMNS : 66
8'ϕ COLUMNS NEAR RAMP E : 10 (FOR ALL LAYOUTS)

8' COLUMNS

FOR COLUMNS NEAR RAMP E, NEGLECT EFFECT OF SCOUR :



SOFFIT Elev ≈ 1195' (Ave)
NEW FILL Elev ≈ 1175'
RIVER BED Elev ≈ 1150'

175' - 30' = 145'

∴ SHAFT = 145 + 25' = 170 FT × \$ 500 = \$85,000

20' HIGH COLUMN = 47 cy × \$ 360 * = \$16,920

\$ 101,920 /COL

* INCLUDES REINFORCEMENT

10' COLUMNS

SHAFT : 110 FT × \$ 750.00 = \$ 82,500

55'ave HIGH COLUMN : 160 cy × \$ 330* = 52,800

\$ 135,300 /COL

COST/ft : 10 × \$ 101,920 = \$ 1,019,200
56 × 135,300 = 7,576,800
\$ 8,596,000 / 419,742 = \$ 20.48

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

PROJECT: SALT RIVER BRIDGE
ITEM: Cost estimate : Substructure
DESIGN: Preliminary
DATE:
SHEET:
OF
REVISION:

180 FT SPANS — TOTAL NUMBER OF COLUMNS = 58

10' COLUMNS : SHAFT : 120' × $750 = $ 90,000
COLUMN : 160 cy × $ 330 = 52,800
$ 142,800

COST/ф : 10 × $ 101,920 = $ 1,019,200
48 × 142,800 = 6,854,400
$7,873,600 / 419,742 = $ 18.76

200 FT SPANS — TOTAL NUMBER OF COLUMNS = 50

10' COLUMNS : SHAFT : 130' × $750 = $ 97,500
COLUMN : 160 cy × 330 = 52,800
$ 150,300

COST/ф : 10 × 101,920 = $ 1,019,200
40 × 150,300 = 6,012,000
$ 7,031,200 / 419,742 = $ 16.75

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

PROJECT: SALT RIVER BRIDGE
ITEM: Cost estimate : substructure
DESIGN:
DATE:

SHEET:
OF
REVISION:

240 FT SPANS   TOTAL NUMBER OF COLUMNS : 46

10' COLUMNS : SHAFT : 175' × $750 = $131,250

COLUMN : 160 cy × $330 = 52,800

$184,050

COST/⌀ = 10 × $101,920 = $1,019,200

36 × 184,050 = 6,625,800

$7,645,000 / 419,742 = $18.21

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
1817 N. 7th St., Suite 270, Phoenix, AZ 85006

PROJECT: E. Papago Sec 6
ITEM: Salt River
DESIGN: Quantities 160 ft
DATE: 08-02-89 FP

SHEET: OF
REVISION:

References

Superstructure: (D=7.5')
Quantities for 1' Section.

Pier Diaph.	(319.83+13.61)(10')/27	0.77 cy
Intermediate Diaph.	(344.71+16.04)(0.75')/27	0.083 cy
Hinge Diaph.	(291.12+16.04)(6')/27	0.43 cy
Typ. Section	(169.24')(1')/27	6.27 cy
Soffit Haunch	L= 56.83' 2(1/2)(56.83')(17.67)(6"/12)/27	0.12 cy
Web Flares.	(10)(20')(5"/12)(6.33')/27	0.12 cy
		$\boxed{7.793 \text{ cy}}$
Prestress	(10)(4)(18)(3.4)(0.217)(1') =	531.22 lbs
Resteel	(200 #/cy)(7.793 cy) =	1558.6 lbs
Concrete	(7.793)($200) =	$1558.6
Rebar	(1558.6)($0.4) =	$623.4
Prestress	(531.22)($1.30) =	$690.6
		$2872.6
		$\boxed{\$34.54/ft}$

PROJECT: E. Papago Sec 6	SHEET:
ITEM: Salt River	OF
DESIGN: Quantities 180 ft	REVISION:
DATE: 08-02-89 FP	

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
1817 N. 7th St., Suite 270, Phoenix, AZ 85006

Superstructure (D = 8.5')
Quantities for 1' Section

Pier Diaph.	(369.20 + 18.68)(10')/27 =	0.8 cy
Intermediate Diaph.	(393.28 + 21.51)(0.75')/27 =	0.064 cy
Hinge Diaph.	(331.22 + 21.51)(6')/27 =	0.44 cy
Typ Section.	(180.69)(1')/27 =	6.69 cy
Soffit Haunch.	(2)(53.71')(17')(6"/12 × 1/2)/27	0.094 cy
Web Flare.	(10)(20)(5/12)(7.33)/27	0.13 cy
		$\boxed{8.218 \text{ cy}}$
Prestress	(10)(4)(18)(3.4)(0.217)(1')	531.22 lbs
Resteel	(200 #/cy)(8.218) =	1643.6 lbs
Concrete	(8.218)($200) =	$1643.6
Rebar	(1643.6)($0.4) =	$657.44
Prestress	(531.22)(1.3) =	$690.6
		$2991.64
		$\boxed{\$35.97/\text{ft}}$

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
1817 N. 7th St., Suite 270, Phoenix, AZ 85006

PROJECT: 534 E. Papago Sec 6.
ITEM: Salt River
DESIGN: Quantities 200 ft
DATE: 08-02-84 FP

SHEET: ____ OF ____
REVISION:

References

Superstructure: (D = 10')

Quantities for 1' Section

Pier Diaphragm	(423.83 + 24.28) x 10' /27	0.83 cy
Intermediate Diaphragm	(502.11 x 0.75')/27	0.07 cy
Hinge Diaphragm	(435.05 x 6) /27	0.48 cy
TYP Section	(189.84)(1')/27	7.03 cy
Soffit Haunch	Length = 83.17' − 2(6' + 3.50) = 64.17' Web Thickness = 10 (14"/12) = 11.67' L = 64.17 − 11.67 = 52.5' (1/2)(52.5')(17.3)(6"/12)/27	0.042 cy
web Flares	(10)(20')(5/12)(8.79')/27 =	0.14 cy
		8.59 cy
Resteel	(8.59 cy)(200 #/cy)	1718 lbs
Prestress	(10)(4)(19)(3.4)(0.217)(1')	561 lbs
Concrete	(8.59)(200) =	$1718
Rebar	(1718)(0.4)	$687.2
Prestress	(561)(1.3)	$729.3
		$3134.5
		$37.69/ft

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
1817 N. 7th St., Suite 270, Phoenix, AZ 85006

PROJECT:	534 E. Papago Sec 6
ITEM:	Salt River
DESIGN:	Quantities 240'
DATE:	08-02-89 FP

SHEET: ______
OF ______
REVISION:

References

Super structure (240' span D = 12')

Quantities for 1' Section

Pier Diaphragm:	(529.62☐ x 10')/27	0.82 CYD
Intermediate Diaphragm	(608.24☐ x 0.75')/27	0.07 CYD
Hinge Diaphragm	(459.94☐ x 6')/27	0.43 CYD
Typ Section:	(213.14☐ x 1')/27	7.9 CYD
Soffit Haunch:	(82.675 − 2(6' + 4.3) = 62.3	
	Web thickness = 11(12/14) = 9.5'	
	Actual Length = L = 62.3 − 9.5 = 52.8'	
	(52.8' x 20' x 6/12)/27	0.08 CYD
Web Flares	(11 x 10.83' x 20' x 5/12)/27	0.15 CYD
		9.45 CYD
Resteel:	(9.5 CYD x 200 #/CYD)	1856 lbs
		44 $/Ft²
Prestress	(10 x 3 x 31 x 0.217 x 3.4 x 1)	687 lbs
Concrete	(9.45 x 200) =	$1890
Rebar	(1890 x 0.4) =	$760
Prestress	(687 x 1.3) =	$893.1
		3543.1
		$42.60/ft

12.5 OTHER SCHEMES CONSIDERED

SEGMENTAL CONSTRUCTION

Construction of the Salt River Bridge by segmental methods is feasible at this site and should be given consideration if falsework is not permitted in the river. Two methods currently in use in this country are balanced cantilever and span-by-span construction. The economic span lengths of this bridge are marginal for balanced cantilever construction; span-by-span would probably be preferred. This method uses a form traveler supported on the piers or an erection truss upon which the precast segments are assembled.

For maximum economy with segmental construction, the box girders should be designed with a minimum number of webs. The width of this bridge would require twin boxes with a CIP closure strip placed between the boxes (similar to Alternate B without the interior webs). The 3^{0}-30′ curves require a slightly wider truss with some articulation desired to accomodate the superelevation.

This bridge is large enough (419,742 square feet) to absorb the higher mobilization costs required for segmental construction. However, local contractors are not familiar with this construction method, nor would they be expected to have the erection trusses and other equipment necessary. as well as the sophisticated construction expertise required to successfully build this type of structure. Segmental bridges also require an overlay wearing surface for smooth rideability.

Construction time can usually be reduced with precast segmental erection. The segments can be fabricated while the substructure is under construction and stockpiled for future erection. The short line casting beds could either be located on or near the site or at a precast plant. The segments could be kept short to permit hauling by special low-boy trailers. The curved alignment of this bridge would require very careful casting control since most of the segments would be different.

Construction management and inspection is critical for successful segmental construction. Construction engineers experienced in this type of bridge construction should be assigned to the project.

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
1817 N. 7th St., Suite 270, Phoenix, AZ 85006

PROJECT:	RAM-600-5-301 D
ITEM:	145' Vs 127' AASHTO Spans
DESIGN:	
DATE:	8/2/89
SHEET:	
OF	
REVISION:	

Comparison of AASHTO Type VI girder spans at 145'
Vs. AASHTO Type VI Modified at 127'.

Use \$100/Lin ft Type VI
\$90/Lin ft. Type VI modified

Delete abutment costs and superst. common items

Bridge Length:
E.B. ℄ Brg. West Abut = 286+25
W.B " " = 288+20

℄ Brg East Abut = 312+40

Length EB = 2615' ÷ 145' = 18 spans 17 piers
WB = 2420 = 17 spans 16 piers > 33 piers

EB = ÷ 127 = 21 spans 20 piers
WB = = 19 spans 18 piers > 38 piers

145' spans
14 girders @ 6'-0" w 7½" deck
14 x 5035' = 70,490'

127' spans
12 girders @ 7'-0" w 8" deck
12 x 5035' = 60,420'

Δ deck concrete
$\frac{.04}{27} \times 83.17' \times 5035' = 620$ cu. yds

Piers	145'	127'
Pier Cap Concrete	= \$1,637,800	
" " Rebar	= 655,120	
Pier Col. Concrete	= 2,217,600	
Pier Rebar	= 945,500	
Drilled Shaft	= 6,042,000	
	11,498,020 × 38/33	= 13,240,144
Girders @ \$100/'	= 7,049,000	@ \$90/' = 5,437,800
Δ Deck Concrete	—	= 124,000
Δ " Rebar	—	= 49,000
	\$18,547,020	\$18,850,944

∴ Use 145' spans

T.Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING
1817 N. 7th St., Suite 270, Phoenix, AZ 85006

PROJECT:	RAM-600-5-301 D
ITEM:	Long Span Unit
DESIGN:	Order of Magnitude Est.
DATE:	8/2/89

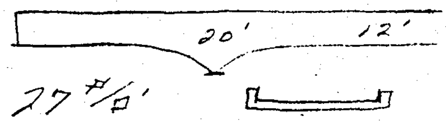
SHEET: ______ OF ______
REVISION:

Incremental Cost of (300'-400'-300') steel plate girder unit over (5@200') unit.

(300'-400'-300') = 57#/□'

(200-200-200-200-200) = 30#/□'

> 27#/□'



Deck Area =

1000 × 83.16 × 2 = 166,320 □'

Incremental Cost (steel only)

Assuming same number of girders: 6 @ 14'-0"/Bridge

27# × 166,320 × 0.90 = $4,041,576

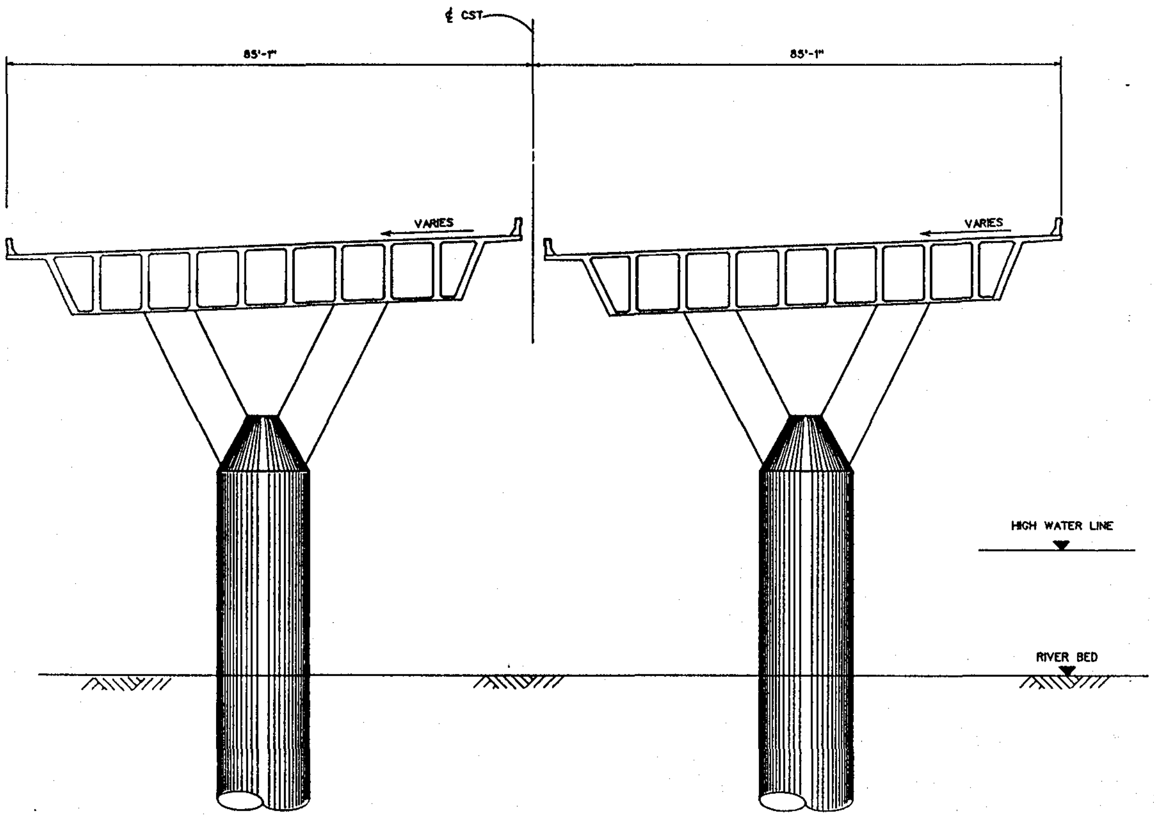
Note: The profile grade would be raised approx. 6' over Hayden Road due to the deeper superstructure.

Substructure

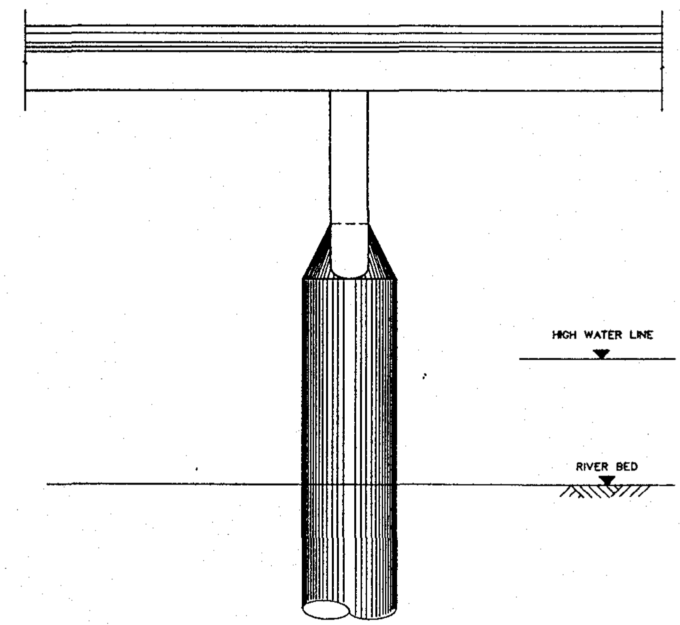
Each Pier for the 200' span alternate is estimated at $154,000. Six 2-column piers would be replaced by four river type piers supported by a footing cap on multiple drilled shafts. The footing would be located below scour, about 50' below streambed. Each of the two piers supporting the midspan are estimated at $450,000. The end span piers would be somewhat smaller and are estimated at $350,000. These four piers would total $1,600,000 versus $924,000 for the six 2-col. piers replaced.

The additional cost of the long span unit vs 200' spans is estimated to range between $4.5 to $5.0 million.

CIP MULTIPLE CELL BOX GIRDER



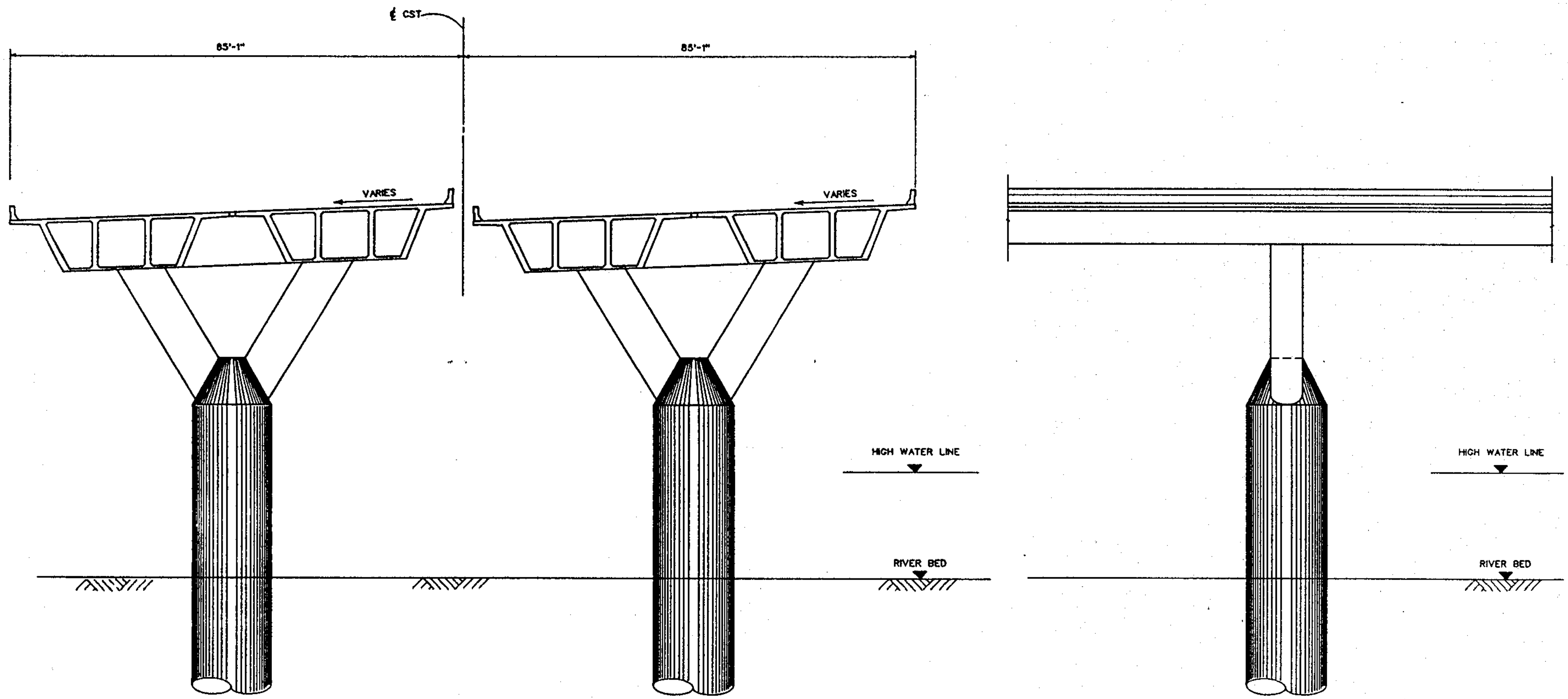
TYPICAL SECTION



ELEVATION AT PIER

SINGLE COLUMN BENT

CIP TWIN BOX GIRDER

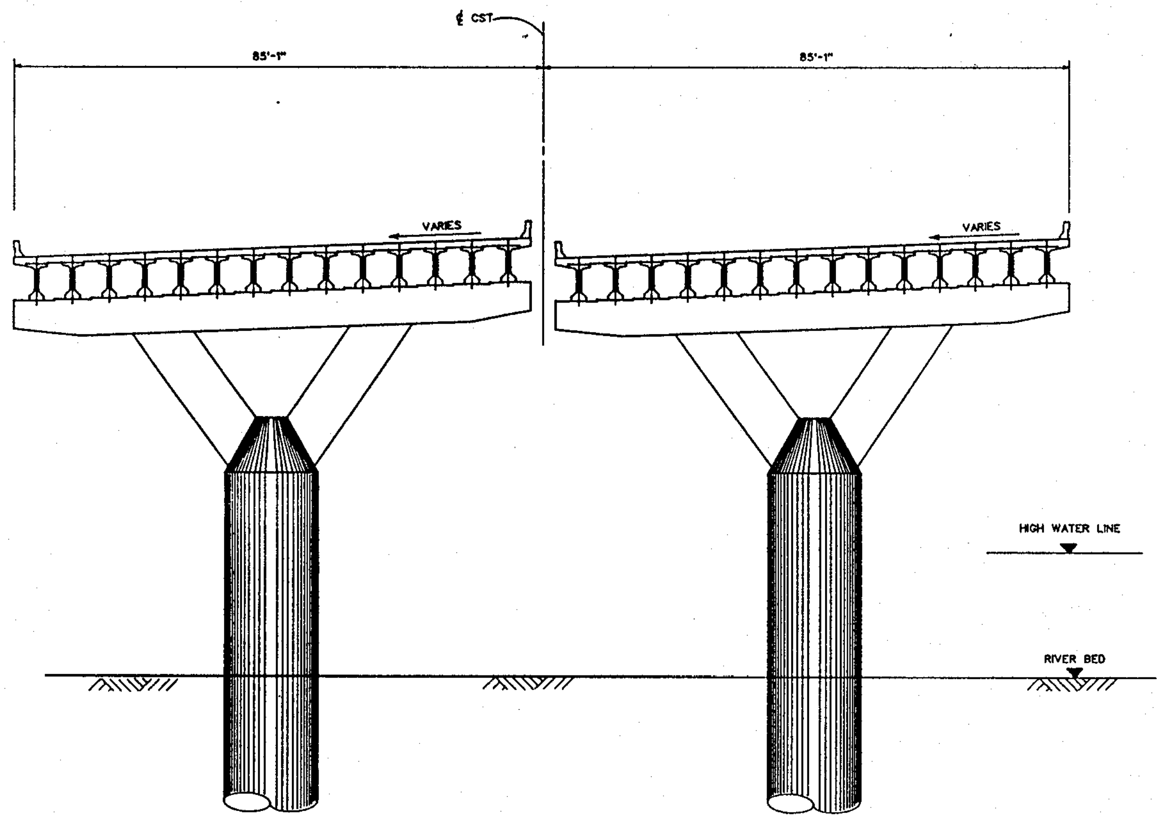


TYPICAL SECTION

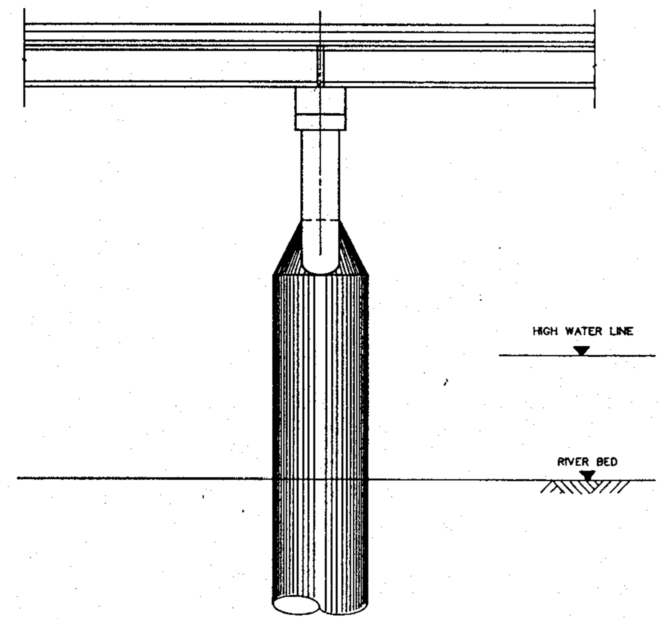
ELEVATION AT PIER

SINGLE COLUMN BENT

AASHTO VI (145 FT SPAN)



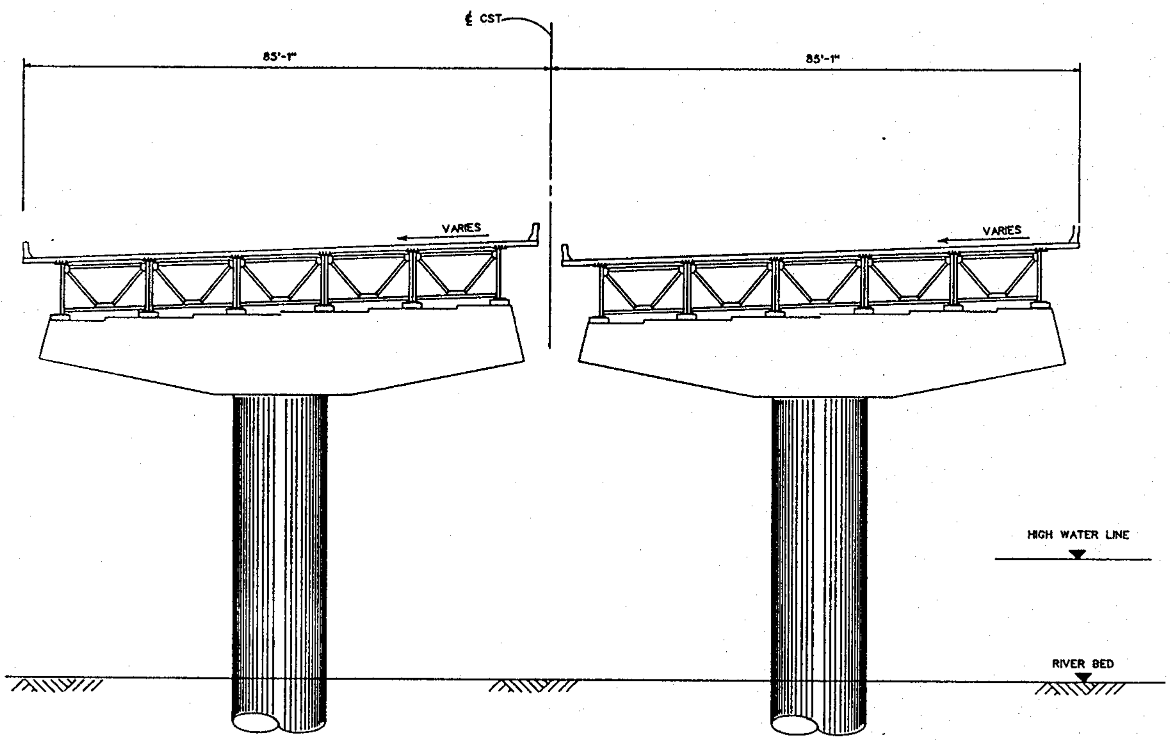
TYPICAL SECTION



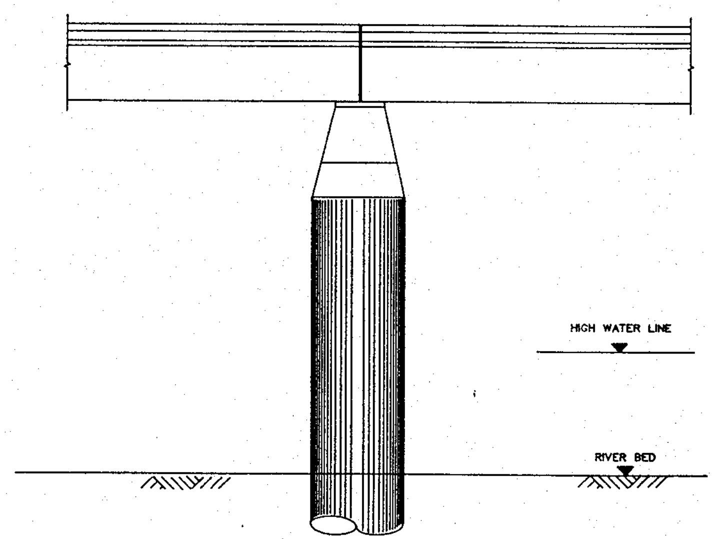
ELEVATION AT PIER

SINGLE COLUMN BENT

STEEL PLATE GIRDER



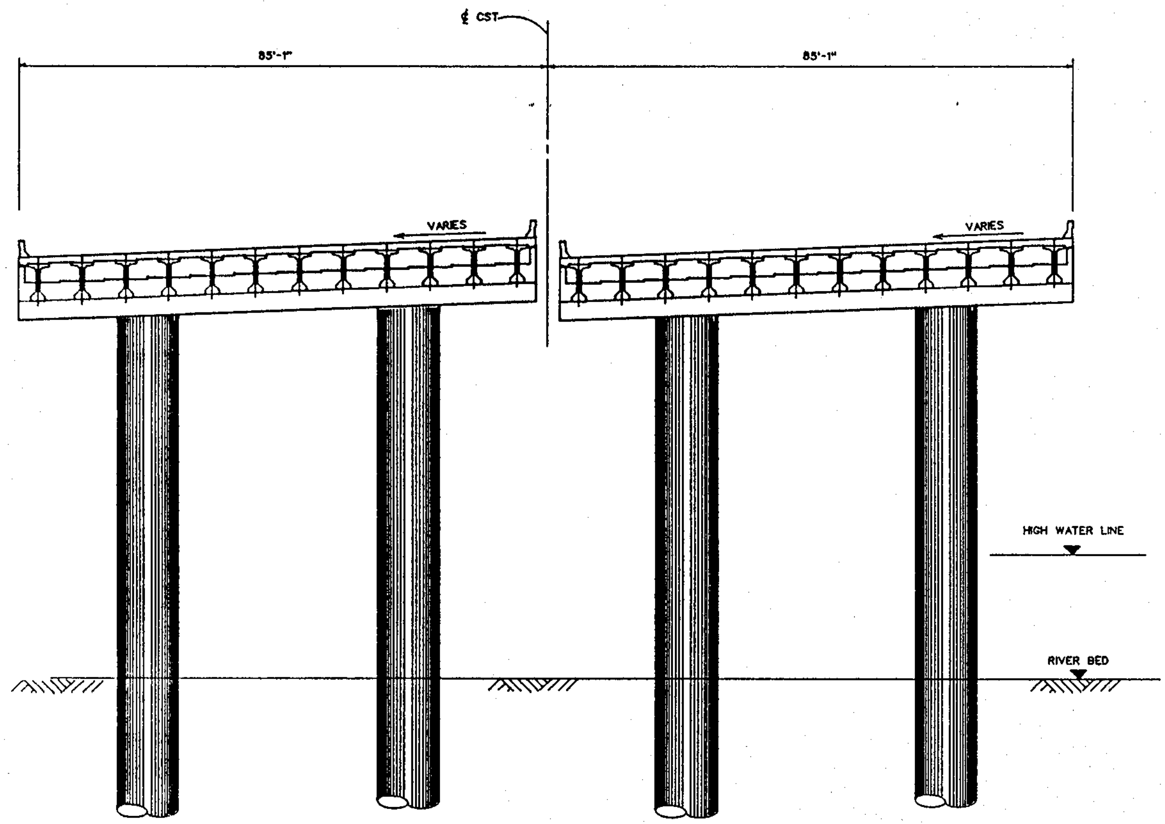
TYPICAL SECTION



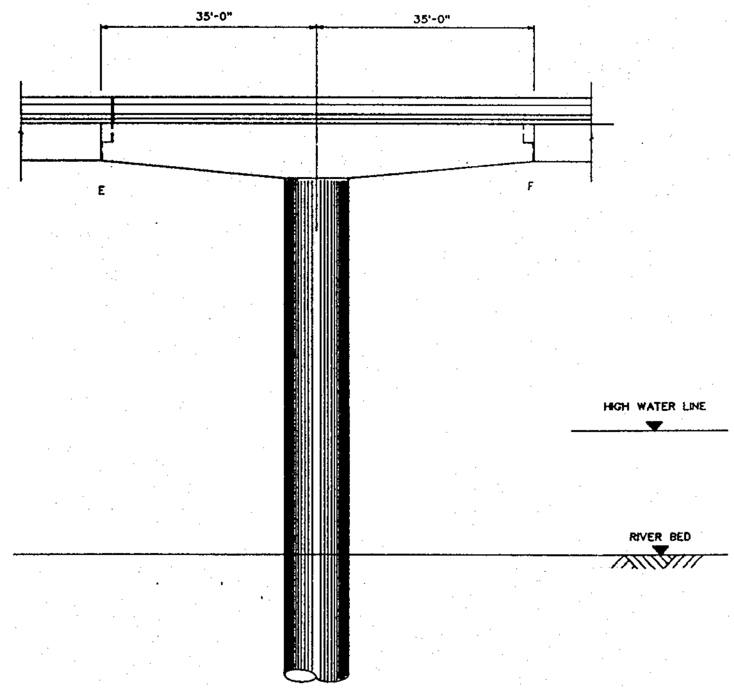
ELEVATION AT PIER

SINGLE COLUMN BENT

AASHTO VI : DROP-IN-SPAN



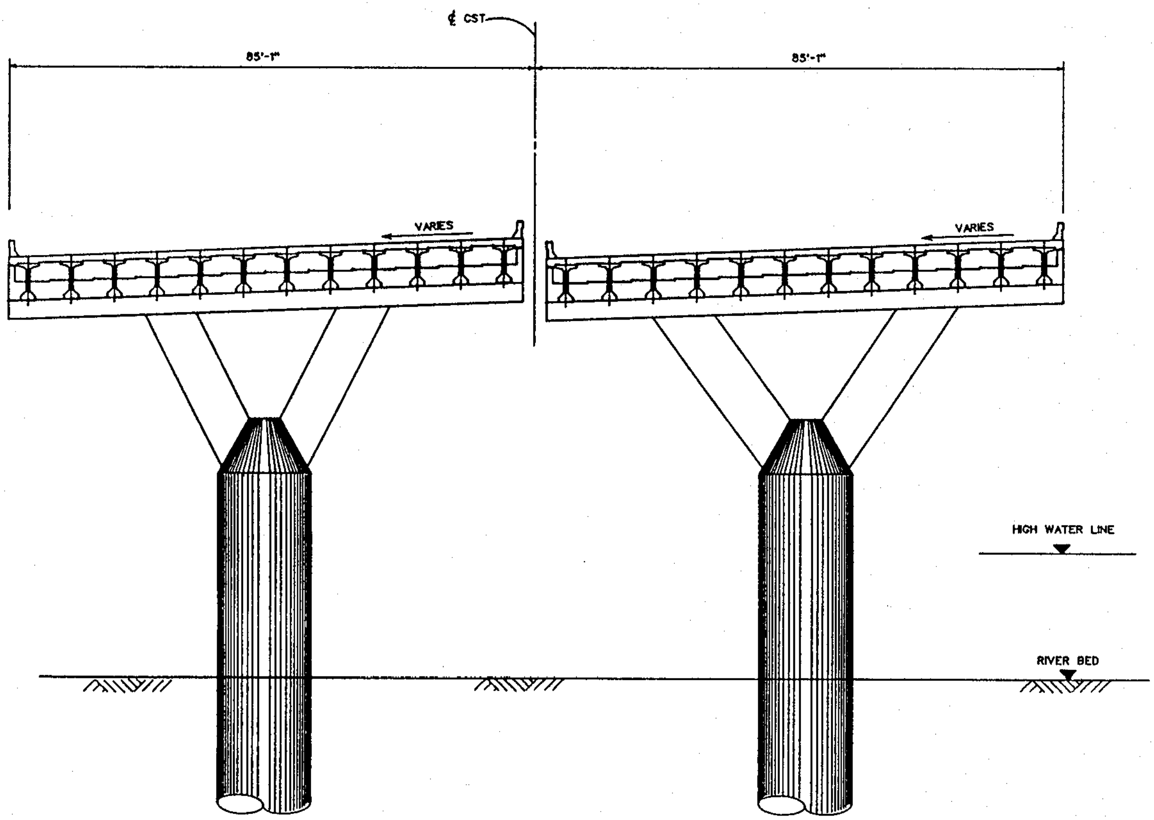
TYPICAL SECTION



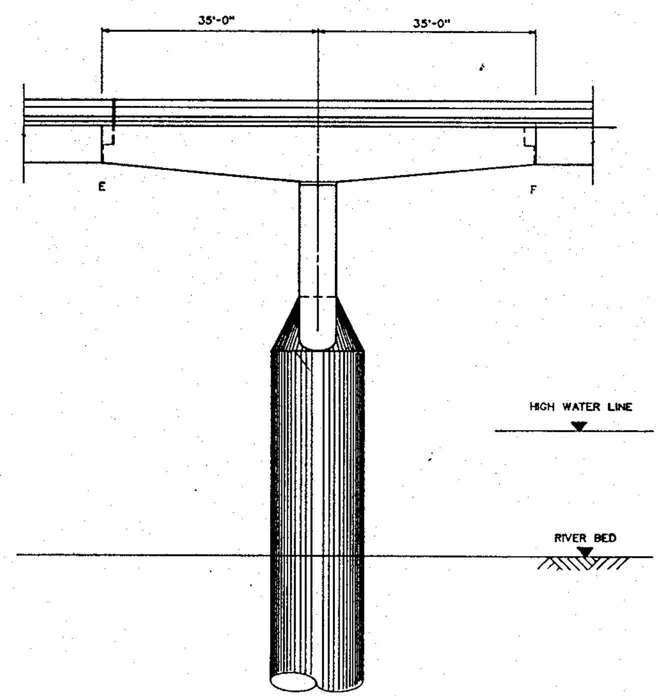
ELEVATION AT PIER

TWO COLUMN BENT

AASHTO VI : DROP-IN-SPAN



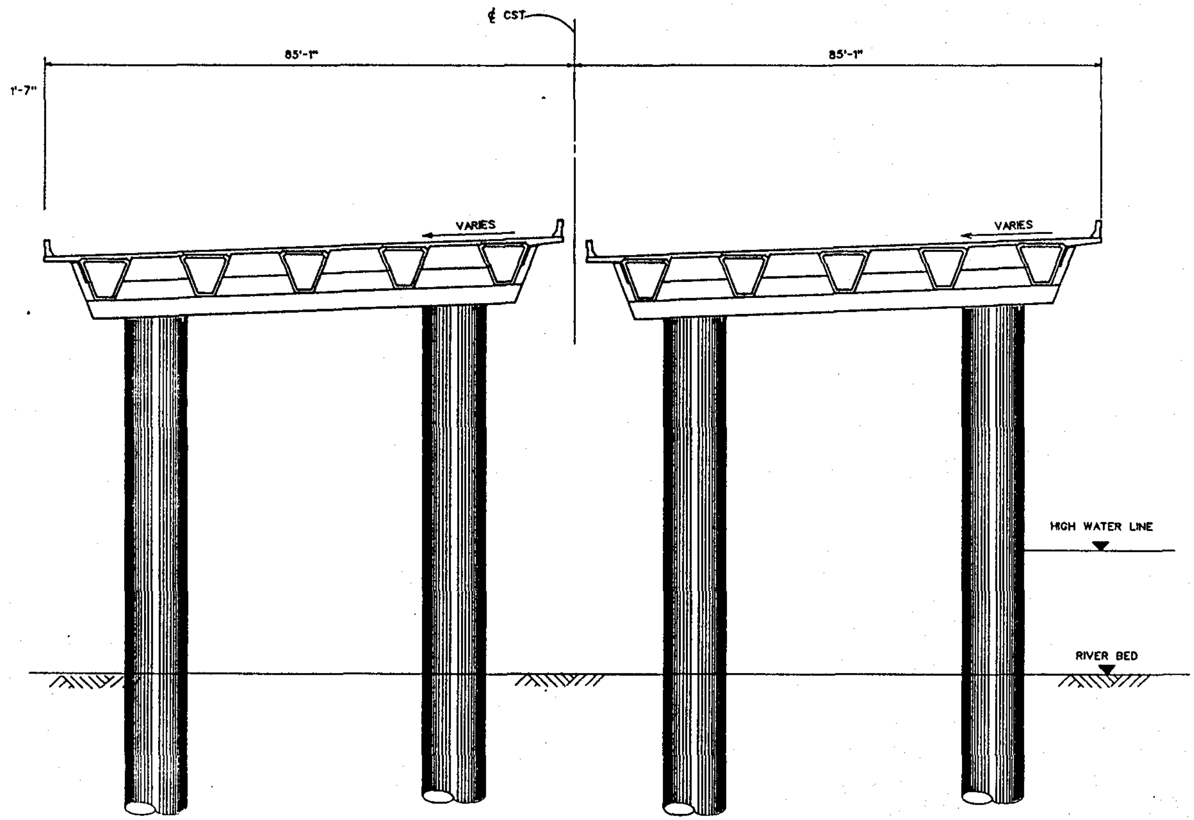
TYPICAL SECTION



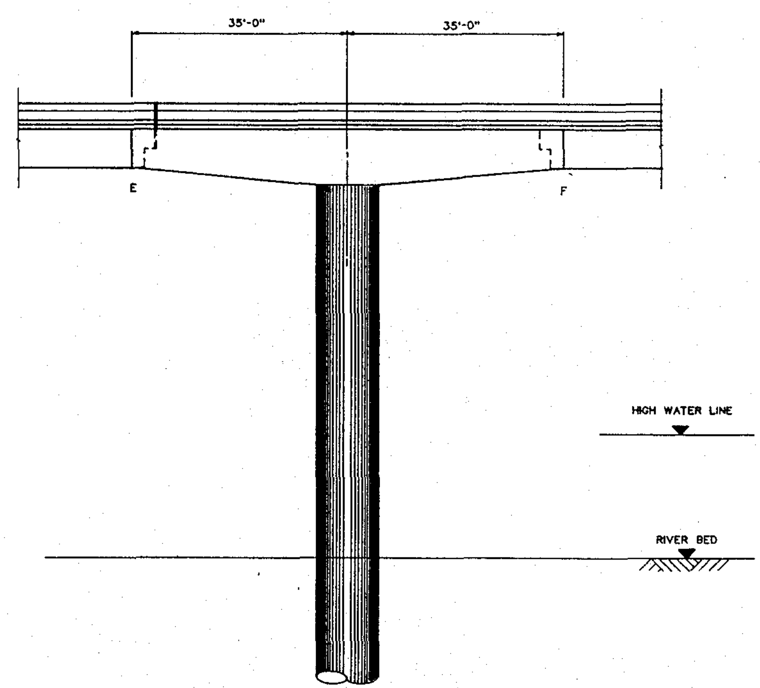
ELEVATION AT PIER

SINGLE COLUMN BENT

PRECAST BOX : DROP-IN-SPAN



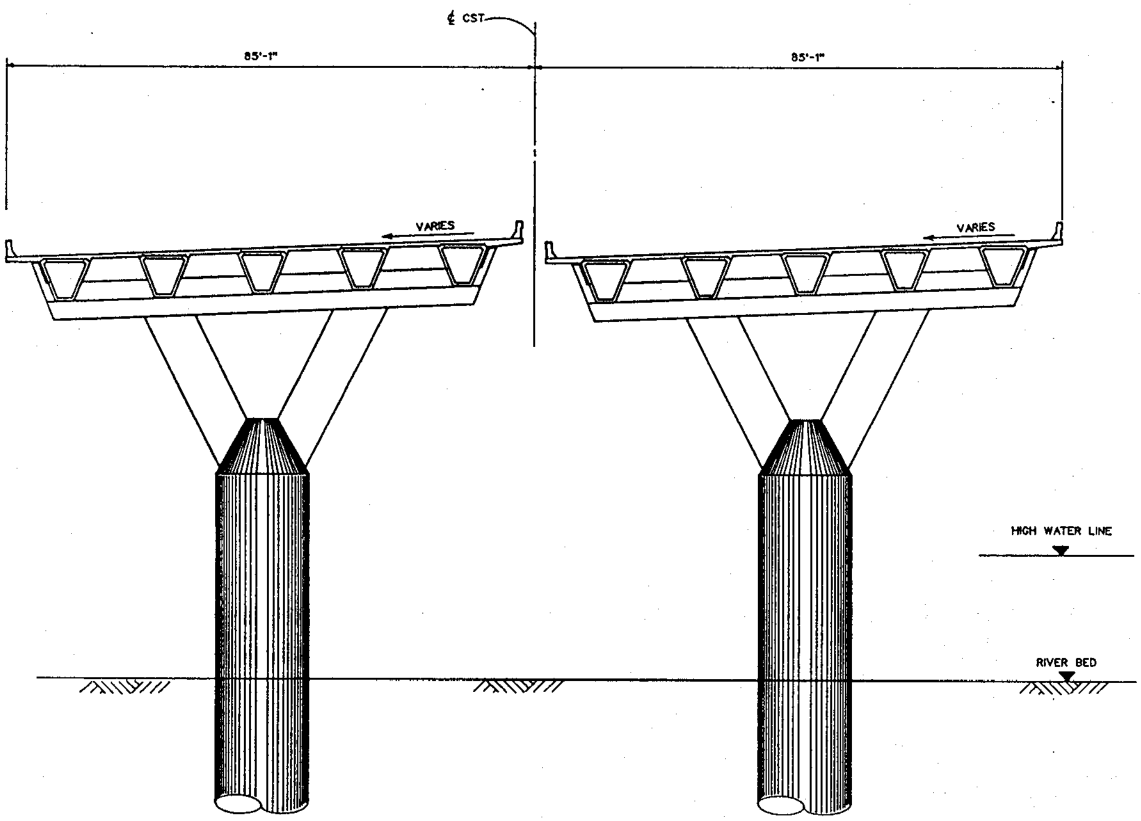
TYPICAL SECTION



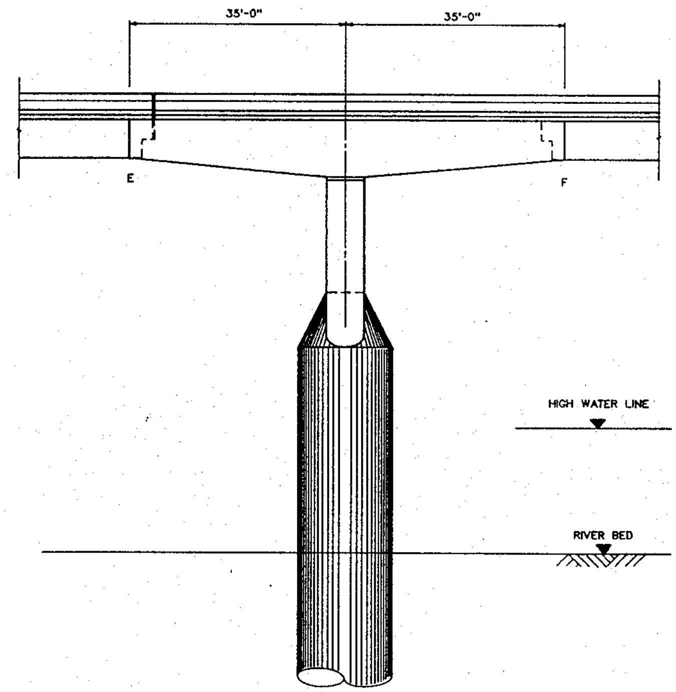
ELEVATION AT PIER

TWO COLUMN BENT

PRECAST BOX - DROP-IN-SPAN



TYPICAL SECTION

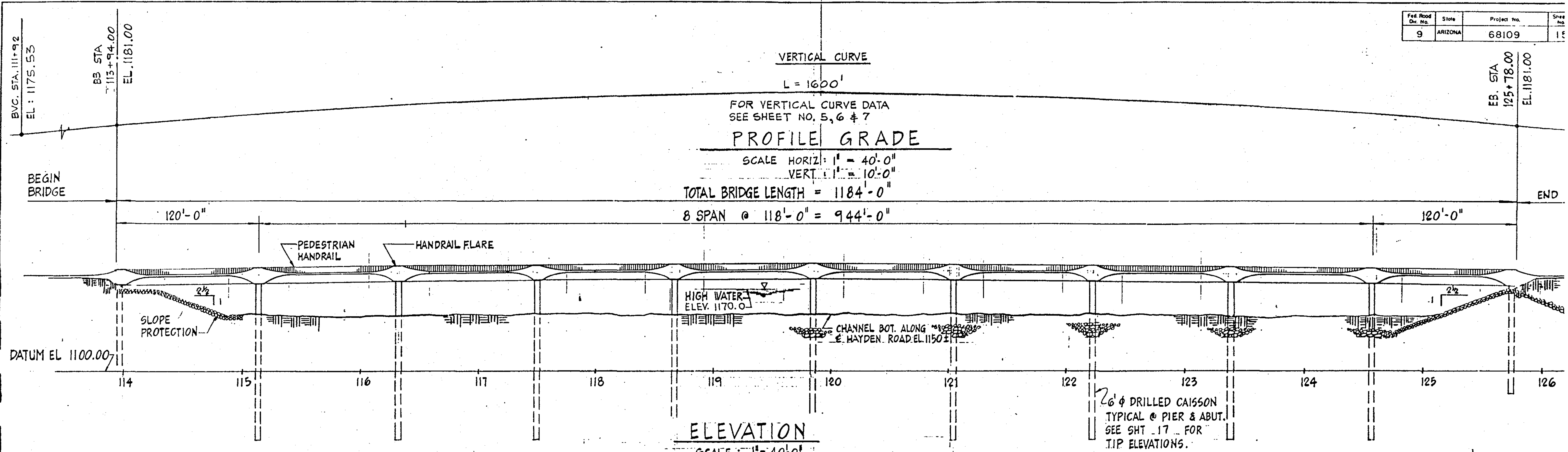


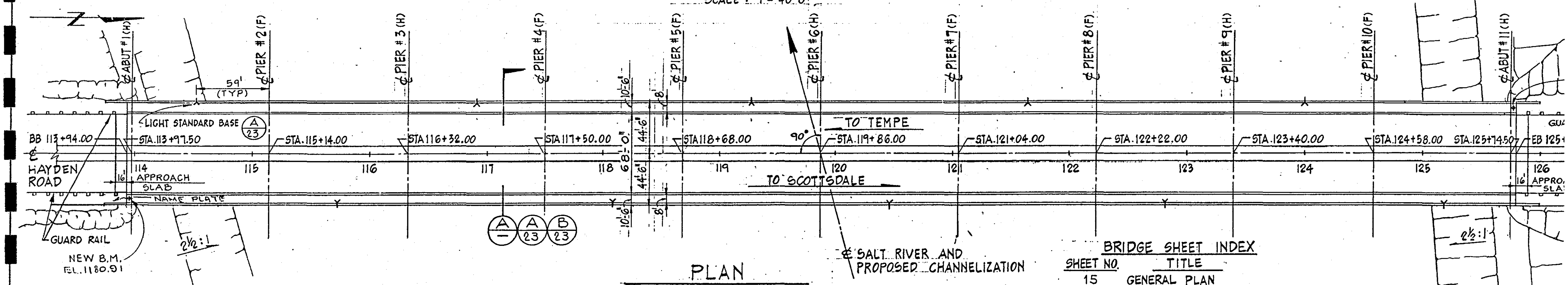
ELEVATION AT PIER

SINGLE COLUMN BENT

12.6 HAYDEN ROAD BRIDGE RECORD PLANS

Fed. Road Div. No.	State	Project No.	Sheet No.
9	ARIZONA	68109	15





PLAN

SCALE 1" = 40'-0"

(H) DESIGNATES HINGED ABUTMENT OR PIER.

(F) DESIGNATES FIXED PIER. (SEE DETAILS).

LIVE LOADING = HS20-44

STREAM FLOW = 200,000 CFS

GENERAL NOTES:

SEE SHEET NO. 17.

REFERENCE BENCH MARK:

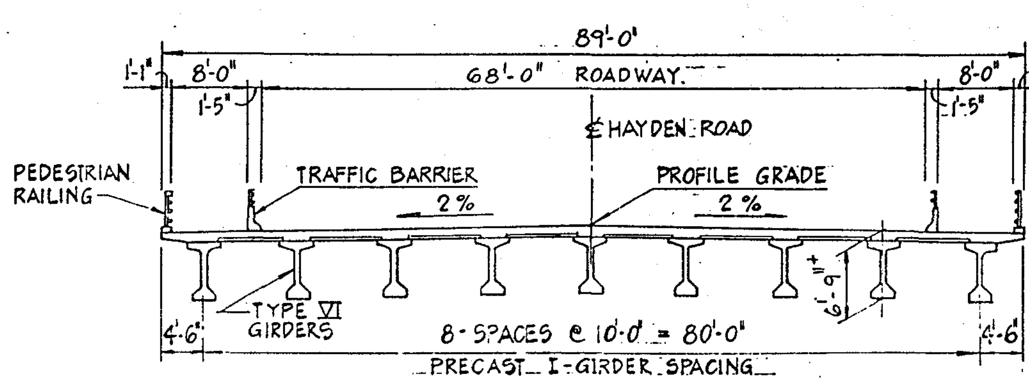
M.C.H.D. DATUM: B.M. @ E¼ SEC. 14 STA. 100+00 WEST EDGE OF H.H. RING EL. 1172.23

BASIS OF BEARING @ ℄ HAYDEN ROAD:

CONSTRUCTION ℄ PARALLEL TO ½ LINE. ½ LINE: EASTERLY E¼ SECTION LINE N 04°06'20" W.

⊕ NEW BENCH MARK:

B.M. = BRASS PLUG FURNISHED BY M.C.H.D. & SET BY CONTRACTOR.



TYPICAL SECTION

SCALE: 1" = 10'-0" PRECAST SLAB PANEL ALTERNATE

BRIDGE SHEET INDEX

SHEET NO.	TITLE
15	GENERAL PLAN
16	FOUNDATION PLAN
17	6' DIA. CAISSON
18	ABUTMENT PLAN & ELEVATION
19	ABUTMENT DETAILS
20	FIXED PIER PLAN & ELEVATION
21	HINGED PIER PLAN & ELEVATION
22	PIER SECTIONS & DETAILS
23	TYPICAL SECTION
24	GIRDER LAYOUT NO. 1
25	GIRDER LAYOUT NO. 2
26	GIRDER DETAILS.
27	DECK ELEVATIONS
28	MISC. DETAILS & SECTIONS.
29	BARRIER RAILING DETAILS.

RECORD DRAWIN

14143 DAVID JOSEPH ARIZONA, U.S.A.

MARICOPA COUNTY HIGH

M.C.H.D. PROJECT NO. 68109

M.C.H.D.

HAYDEN ROAD BR

GENERAL PLA

Boyle Engineering Corpo

consulting engineers / arc

DESIGN D.S.

DRAWN V.C.

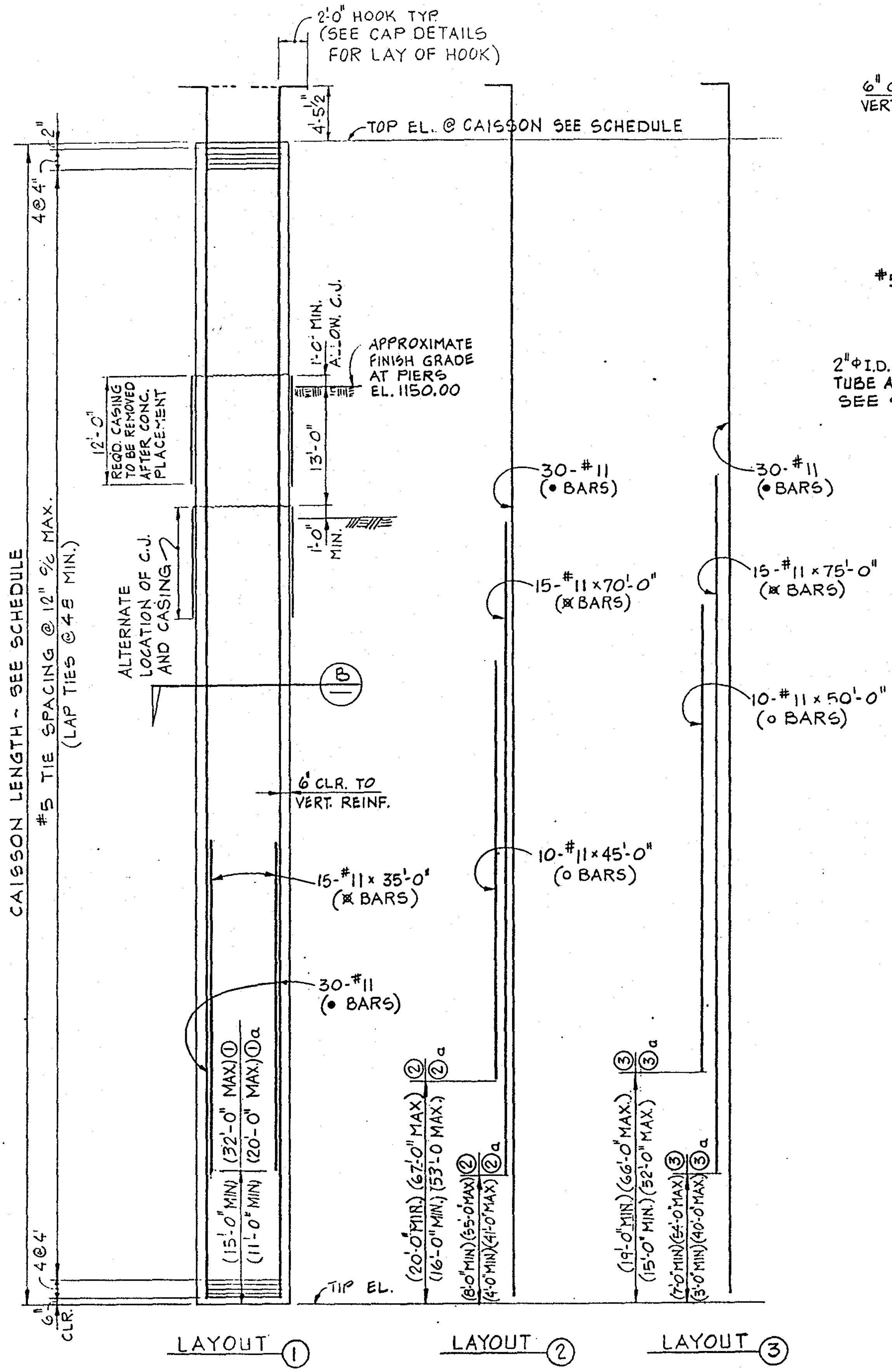
CHECKED J.B.

DATE JUNE '82

PH-M04-101-01 B-5

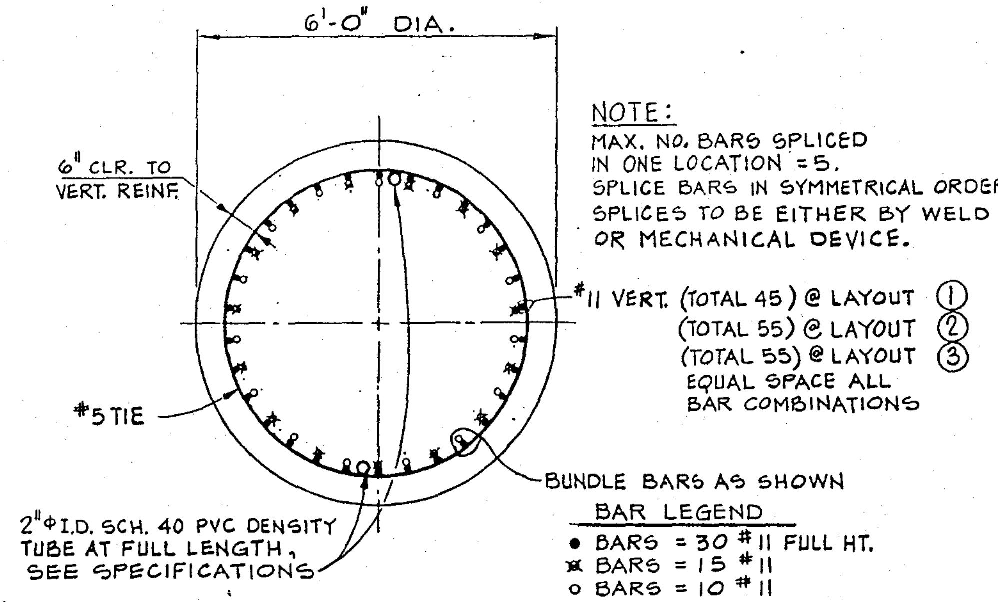
REV	DATE	DESCRIPTION	APP

Fed. Road Div. No.	State	Project No.
9	ARIZONA	68109



6' DIA. CAISSON ELEVATION & REINFORCING (A/17)

SCALE: NONE



NOTE:
MAX. NO. BARS SPLICED IN ONE LOCATION = 5.
SPLICE BARS IN SYMMETRICAL ORDER.
SPLICES TO BE EITHER BY WELD OR MECHANICAL DEVICE.

BAR LEGEND
- • BARS = 30 #11 FULL HT.
- ⌘ BARS = 15 #11
- o BARS = 10 #11

SECTION (B)

SCALE: NONE

6'-0" DIA. CAISSON ELEVATION-SCHEDULE

LOCATION & CAISSON NO.	TIP ELEVATION MINIMUM	TIP ELEVATION MAXIMUM	REINFORCING LAYOUT	TOP ELEVATION
A-1: EXT. / CTR.	1059.00 / 1055.00	1050.00 / 1038.00	①a / ①	1168.64 / 1169.30
P-2: EXT. / CTR.	1046.00 / 1042.00	1009.00 / 995.00	②a / ②	1171.07 / 1171.73
P-3: EXT. / CTR.			③a / ③	1172.77 / 1173.43
P-4: EXT. / CTR.			③a / ③	1174.21 / 1174.87
P-5: EXT. / CTR.			③a / ③	1174.99 / 1175.65
P-6: EXT. / CTR.			③a / ③	1175.12 / 1175.78
P-7: EXT. / CTR.			③a / ③	1174.99 / 1175.65
P-8: EXT. / CTR.			③a / ③	1174.21 / 1174.87
P-9: EXT. / CTR.			③a / ③	1172.77 / 1173.43
P-10: EXT. / CTR.			②a / ②	1171.07 / 1171.73
A-11: EXT. / CTR.	1059.00 / 1055.00	1050.00 / 1038.00	①a / ①	1168.64 / 1169.30

*TOP ELEV. = ℄ CAISSON AT 2" ABOVE SOFFIT OF ABUTMENT OR PIER.

NOTE:
TIP ELEVATIONS ARE ESTIMATED MINIMUM AND MAXIMUM DEPTH ELEVATIONS DEPENDENT UPON ACTUAL MATERIAL EXCAVATED AND OBSERVED. FINAL TIP ELEVATIONS WILL BE DETERMINED IN THE FIELD BY THE ENGINEER. APPROXIMATE BRIDEGE QUANTITIES ARE BASED UPON MAXIMUM DEPTH TIP ELEVATIONS.

GENERAL NOTES

1. DESIGN SPECIFICATIONS: AASHTO 1977 WITH INTERIM SPECIFICATIONS.
2. LOADINGS: LIVE = HS20-44, FUTURE DECK WEARING SURFACE = 25 PSI
3. REINFORCED CONCRETE: CAISSONS - SEE CAISSON NOTES BELOW.
   ABUTMENT AND PIER CAPS f'_c = 3500 psi
   PRESTRESS GIRDERS f'_c = 5000 psi
   CONCRETE DECK f'_c = 3500 psi
   PRECAST CONCRETE DECK SLABS f'_c = 5000 p
   ALL CONCRETE SHALL BE CLASS "AA"
4. BAR REINFORCING STEEL: ASTM A-615 GRADE 60 - ALL.
5. ALL EXPOSED METAL SHALL BE GALVANIZED AFTER FABRICATION.
6. GROUT FOR BEARINGS AND RAILING POSTS: NON-SHRINK AND NON-MI
7. FUTURE UTILITY OPENING SITES SHALL BE VERIFIED PRIOR TO CONST
8. REFER TO SHEET NO.30 FOR FOUNDATION AND SOILS DATA.
9. ~~SOIL TEST BORINGS FOR THIS PROJECT WERE MADE DURING DECEMBE THEIR APPROXIMATE LOCATION IS MARKED ON THE FOUNDATION PLAN SYMBOL. SUBSURFACE SOIL INVESTIGATION RESULTS ARE AVAILABL OFFICE OF THE COUNTY ENGINEER, MARICOPA COUNTY HIGHWAY DEPARTM ARE FOR INFORMATION ONLY, IN ACCORDANCE WITH THE STANDARD SPI AND NO WARRANTY IS MADE THEREFOR.~~
10. CONSTRUCTION SPECIFICATIONS: UNIFORM STANDARD SPECIFICATIONS WORKS CONSTRUCTION. MAG DATED THE SPECIAL PROVISION.

6' Ø DRILLED CAISSON NOTES

1. OVERHEAD AND BELOW GRADE UTILITIES AND EXISTING FACILITIES TO SHALL BE LOCATED PRIOR TO CAISSON EXCAVATION.
2. VERTICAL ALIGNMENT OF COMPLETED CAISSONS SHALL BE WITHIN 1.0 FROM FINISH GRADE TO TIP AND WITHIN 0.2 FEET OF PLAN LOCATION ELEVATION 1150.0.
3. FINAL OUTSIDE DIAMETER OF THE DRILLED EXCAVATION SHALL BE W TO + 9.0 INCHES FROM THE SPECIFIED CAISSON DIAMETER.
4. USED OR WASTED DISPLACEMENT SLURRY SHALL NOT BE ALLOWED T FLOWING WATER OF THE SALT RIVER.
5. CAISSON CONCRETE SHALL BE PLACED BY PUMP AND TREMIE METHO
6. CONCRETE STRENGTH: BELOW GRADE: f'_{cl} = 4500 psi AT 28 DAYS
   ABOVE GRADE AND FORMED: f'_{cl} = 4000 psi A
7. BAR REINFORCING STEEL: ASTM A-615 GRADE 60.
8. UPPER LIMIT OF DRILLED CAISSONS
   ABUTMENT 1 & 11 - EL. 1168.0
   PIER 2-10 - EL. 1150.0

RECORD DRAW

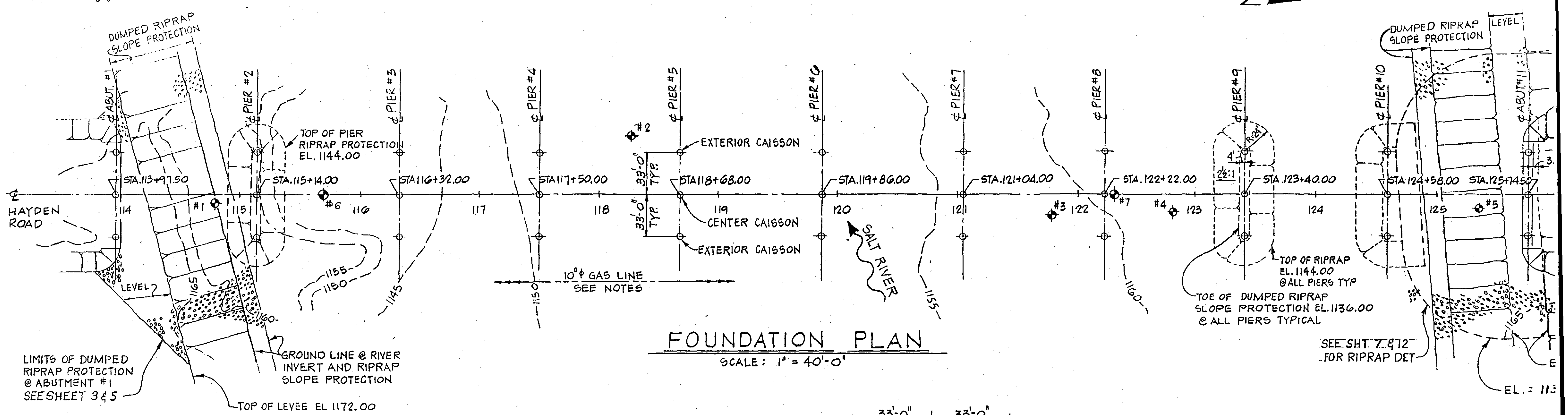
DAVID JOSEPH NOICHL 14143

MARICOPA COUNTY H
M.C.H.D. PROJECT NO. 68109

HAYDEN ROAD
6' DIA. CAISS
DETAILS & N

DESIGN D.S. DRAWN W.C. CHECKED J.B.
DATE JUNE '82
PH-M04-10F01
Bouls Engineering Co. consulting engineers
REV. DATE DESCRIPTION APP.

Fed. Road Div. No.	State	Project No.
9	ARIZONA	68109

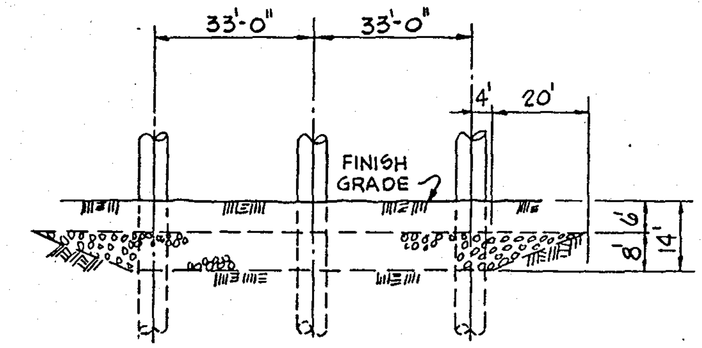


FOUNDATION PLAN

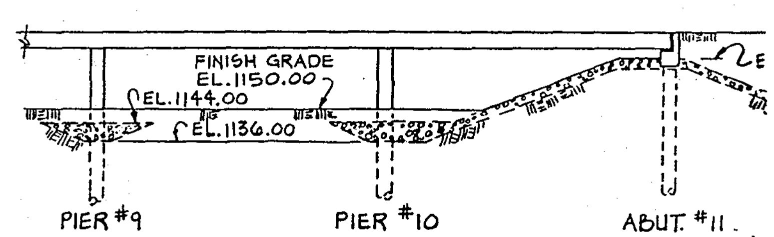
SCALE: 1" = 40'-0'

LEGEND

- ------ EXISTING CONTOURS @ AUGUST, 1980
- 6'Φ CAISSON
- OVERHEAD POWER LINE SUPPORT STRUCTURE
- –G–G– 10"Φ BELOW GRADE GAS LINE
- SOILS TEST BORING AND NUMBER



TYPICAL SECTION
DUMPED ROCK RIPRAP @ PIERS
SCALE: 1" = 20'



SECTION ALONG ¢ HAYDEN ROAD
DUMPED ROCK RIPRAP
SCALE: 1" = 40'

TYPICAL DUMPED ROCK RIPRAP SECTIONS

APPROXIMATE BRIDGE QUANTITIES				
ITEM	DESCRIPTION	UNIT	QUANTITY	
1	STRUCTURE BACKFILL	C.Y.	140	
2	ABUTMENT & PIER CONCRETE $f'_c = 3500$	C.Y.	1,415	(CLASS AB)
3	DECK, DIAPHRAGMS, $f'_c = 3500$ ALT. # B	C.Y.	2,875	(CLASS AB)
4	DECK, DIAPHRAGMS, $f'_c = 3500$ ALT. # A	C.Y.	3,440	(CLASS AB)
5	PRECAST DECK PANEL - ALT. # B	S.F.	65,900	
6	FIXED DIAPH CONCRETE, $f'_c = 5000$ psi	C.Y.	510	(CLASS X)
7	PRECAST HANDRAIL FLARES	EA.	40	
8	ROADWAY APPROACH SLAB, $f'_c = 3500$	C.Y.	88	(CLASS AB)
9	REINFORCING STEEL - ALT. # B	LB.	961,000	
10	REINFORCING STEEL - ALT. # A	LB.	1,203,000	
11	METAL BARRIER RAILING	L.F.	2,368	
12	PEDESTRIAN HANDRAILING	L.F.	2,208	
13	DRILLED FOUNDATION CAISSONS $f'_c = 4500$	L.F.	4,557	
14	CAISSON / COLUMN EXTENSION $f'_c = 4000$	L.F.	755	(CLASS AA)
15	PRESTRESSED GIRDERS	EA.	90	
16	LIGHTING CONDUIT & ACCESSORIES	L.F.	2,452	

RECORD DRAWIN

MARICOPA COUNTY HI
M.C.H.D. PROJECT NO. 68109
HAYDEN ROAD E
FOUNDATION I

DESIGN D.S.
DRAWN W.C.
CHECKED J.B.
DATE JUNE '82
PH-M04-101-01

Bouia Engineering Co. consulting engineers

REV | DATE | DESCRIPTION | APP

sla

SIMONS, LI & ASSOCIATES, INC.

4600 S. MILL AVENUE, SUITE 280
TEMPE, ARIZONA 85282
TELEPHONE (602) 491-1393

Dennis L. Richards, P.E.
Assistant Vice President

June 7, 1989

Mr. Raymond L. Cox, Jr., P.E.
Alpha Engineers Inc.
2701 East Camelback, Suite 250
Phoenix, Arizona 85016-4306

Re: Preliminary Hydraulic Design Parameters for the East Papago Freeway Crossing of Indian Bend Wash

Dear Mr. Cox:

Provided herewith is information you requested during our May 25, 1989 meeting regarding the hydrology and hydraulics of the East Papago crossing of Indian Bend Wash.

The flow rates recommended by the U.S. Army Corps of Engineers (COE), Los Angeles District, in their general design memorandum for Indian Bend Wash, May 1975, are given below in Table 1.

TABLE 1. INDIAN BEND WASH DISCHARGE FREQUENCY VALUES

Recurrence Interval (years)	Discharge (cfs)
10	8,000
50	21,000
100	30,000
SPF	62,000

The Indian Bend Wash channel is designed to convey the 100-year flood of 30,000 cfs. The water-surface elevation at the East Papago crossing of Indian Bend Wash will likely be affected by flow in the Salt River. The 100-year peak discharge for the Salt River is 215,000 cfs. The estimated 100-year water-surface elevations at the East Papago crossing of Indian Bend Wash are provided in Table 2.

TABLE 2. ESTIMATED 100-YEAR WATER SURFACE ELEVATIONS

Cross-Section Number[1]	Stream Distance (ft)	Calculated Water Surface Elevation Existing (ft)	w/Freeway (ft)
100.00	0.	1167.71	1167.99
200.00	360.	1167.78	1168.05
201.00	570.	1167.99	1168.11
300.00	930.	1168.26	1168.36

[1] Cross-section 100.0 is the confluence of the Salt River and Indian Bend Wash. Cross-sections 200 and 201 are the upstream and downstream faces of the modeled structure.

The assumptions made in the analysis were as follows:

1. The bridge superstructure would remain above the 100- year water-surface elevation leaving only pier area to obstruct the flow.

2. The piers are assumed to be six feet in diameter on center with 130-foot spans.

3. The piers are aligned to the flow with a skew of approximately 38 degrees right with respect to the centerline of the East Papago Freeway.

We would recommend that the pier cap not extend below the 100-year water-surface elevation if the pier cap is wider than the pier itself. A design of this type would increase the effective area of the pier, which would increase the scour potential during the high frequency events and create more severe debris problems at the structure. Furthermore, the Flood Control District of Maricopa County (FCD) is responsible for Indian Bend Wash and preliminary contact with the FCD indicates that they may require three feet of freeboard at the Indian Bend Wash crossing. Based on this preliminary analysis, this would require the low chord elevation to be no less than 1171 feet.

Local scour at the 6-foot diameter piers is estimated to be 18 feet. With 6-feet of debris build-up added to each pier, local scour is estimated to be 29 feet. The flow velocities at the bridge are estimated to be 11 to 12 feet per second (fps).

As mentioned previously, the East Papago Freeway crossing of Indian Bend Wash is affected by the Salt River. From previous studies conducted by SLA on the Salt River, the general trend is one of degradation. In order to obtain the total scour depths at each pier, 6 feet should be added to the local scour depth. This additional depth is to account for general scour and bed forms.

Preliminary investigations indicate that the design for the Indian Bend Wash outlet channel did not include measures to stabilize the low-flow channel. Therefore, the low-flow channel has the ability to move laterally within the channel. A review of the COE design memorandum indicates riprap protection was designed for the banks of Indian Bend Wash. The riprap was to be placed on a 2.5 to 1 (horizontal to vertical) side slope to a thickness of 15 inches. The riprap was to be buried with landscape fill with a 6 to 1 side slope placed on the revetted levees. A field visit to the site indicated the existence of the riprap at the top of the levees. The riprap was designed to be toed down to a depth equal to the invert of the low flow channel. It is recommended that the abutments of the East Papago Freeway crossing be placed outside the existing bank protection.

If you have any questions regarding the hydrologic or hydraulic data or need additional information, please feel free to contact me.

Sincerely,

SIMONS, LI & ASSOCIATES, INC.

Dennis L. Richards

Dennis L. Richards, P.E.
Assistant Vice President

DLR:klw
PAZ-DMJM-03/PH/L13

cc: Michael Ports, DMJM

Turan Ceran, PE
Vice President

DMJM

RECEIVED
AUG 29 1989
Urban Highway
Section

DOCUMENT NO. 12498
FILE NO. 500.12.1,
626.2, 200.2, 800

August 29, 1989

Copy to
Dennis

Arizona Department of Transportation
205 South 17th Avenue, Room 216E
Phoenix, Arizona 85007

Attention: Mr. Chuck Eaton

Re: Contract No. 86-95
Project Nos. 202L MA H 0858 01D
143 MA H 0843 01D
153 MA H 0880 01D

Subject: Bridge Selection Report
202L MA 151 2151 01C
East Papago (Indian Bend Wash - Loop 101)
Salt River Bridge (Design Section 6)

Dear Mr. Eaton:

Transmitted herewith for your review and approval are four copies of the Bridge Selection Report for the Salt River Bridge (Design Section 6).

We are transmitting by this cover letter copies of this report to ADOT District 1, City of Tempe, SRP, APS, SH&B and SLI.

The 30% design submission is scheduled on October 2, 1989 and the design consultant has requested they be notified of the type selection decision no later than September 14, 1989 to maintain their schedule (see attached letter DN 12389); therefore, we request your review comments as soon as possible.

Daniel, Mann, Johnson, & Mendenhall
300 West Clarendon Avenue
Suite 400
Phoenix, Arizona 85013-3499
Telephone: 602/264-1397

Planning
Architecture
Engineering
Landscape
Architecture

Should you require any additional information or have any questions concerning this submittal, please contact Del Miller at 277-1074.

Very truly yours,

DANIEL, MANN, JOHNSON, & MENDENHALL

For [signature]
Turan Ceran
Project Director

TC:LB/jl

cc: w/o DN 12389
Ron Williams, ADOT District 1 (2 copies)
Harvey Friedson, COT (1 copy)
Anwar Hirany, SH&B (1 copy)
Dennis Richards, SLI (1 copy)
Ray Koffman, SRP (1 copy)
Steve Goodman, APS (1 copy)

Gannett Fleming
OF ARIZONA, INC.
ENGINEERS AND PLANNERS

2411 West Colter
Phoenix, AZ 85015
(602) 433-0480

7975 North Hayden Road
Suite A-105
Scottsdale, AZ 85258
(602) 483-9149

August 24, 1989

12389
626.2
500.12.1, 800

Document No.: 280
File No.: 30.2, 50.9

Mr. Turan Ceran
Project Director
Daniel, Mann, Johnson & Mendenhall
300 West Clarendon Avenue
Suite 400
Phoenix, AZ 85013

E. PAPAGO
TC 2
TM 1
LBS 24-09 Chuck Eaton w/1 copy for review
AL
BM w/copy for review
RJ
MP 4
PW
DM 3
BB 5
FILE 6

Re: Contract No. 88-38
East Papago Freeway
Indian Bend Wash - Loop 101
TRACS No. 202L-MA-OH-0855-01D
Project No. RAM-600-5-301D

Subject: Structure Selection Report
Salt River Bridge

NOTE: Gannett-Fleming to send additional copies of report for review

Dear Mr. Ceran:

In accordance with our contract we are submitting 3 copies of the Structure Selection Report for the Salt River Bridge as it pertains to the above-referenced project.

After consideration of several different bridge types, span layouts and constructability studies, we recommend that the cast-in-place concrete twin box girders (post-tensioned) supported by twin column piers founded on single drilled shafts be selected for the Salt River Bridge.

We feel that the recommended structure best satisfies the project objectives of providing an aesthetically pleasing structure with fewer columns, a structure that can be economically constructed and a structure that has proven low maintenance costs in the Arizona environment.

Gannett Fleming's design schedule is very tight and the design of the Salt River Bridge is on the critical path. The 30% design submittal is currently scheduled for October 2, 1989 and the appropriate structural sheets cannot be prepared until the bridge type, size and location is finalized. Therefore, in order to maintain the current design schedule, the bridge type selection is scheduled for no later than September 14, 1989.

DMJM
EAST PAPAGO PROJECT
AUG 24 1989
RECEIVED ☒ SENT ☐

In addition to the design schedule requirements discussed above, the delays encountered by the MC in completing the preliminary geotechnical program are now impacting Gannett Fleming's design schedule.

We are available to meet with you as necessary to accelerate the design and provide information that will assist in the decision making process. If you have any questions, please call.

Very truly yours

GANNETT FLEMING OF ARIZONA, INC.

Terry L. Koons

Terry L. Koons, P.E.
Project Manager

TLK/rmr

* NOTE: SEE SHEET 1 OF 9 FOR CURVE DATA

PRELIMINARY
NOT FOR CONSTRUCTION

DATE	DESCRIPTION	BY	DESIGN T.O / DRAWN J.C.P. / CHECKED F.J. / APPROVED P.P.
REVISIONS			

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
EAST PAPAGO - HOHOKAM - SKY HARBOR

DMJM IN ASSOCIATION WITH H. W. LOCHNER, INC.

SECTION 6 GENERAL PLAN

SCALE: 1" = 50' | DATE: 14 FEB 1989 | DWG. NO. 2 OF 9

___ OF ___